SALT PONDS
INLET MANAGEMENT PLAN

For:

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Department of Public Works
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SALT PONDS INLET MANAGEMENT PLAN
Table of Contents

EXHIBITS, TABLES & FIGURES

REPORT EXHIBITS

REPORT TABLES

APPENDIX A – SUPPLEMENTARY FIGURES

1. EXECUTIVE SUMMARY

1.1 Project Purpose and Background

The objective of the Salt Ponds Inlet Management Plan (IMP) is to develop a plan for engineering improvements and other management actions designed to reduce the cost of maintenance dredging the City maintained portion of the Salt Ponds Inlet. The engineering measures recommended are designed to reduce shoaling thereby reducing the frequency and volume of maintenance dredging and other management actions to contain cost are also included. A major component of this study is a dynamic model designed to gain a better understanding of the inlet's hydraulics and local sediment transport dynamics in order to improve the reliability of recommendations for engineering actions to reduce shoaling. The study analyzed a series of engineering alternatives intended to reduce the frequency of maintenance dredging by use of an integrated computational modeling approach. The approach included two-dimensional simulations of sedimentation patterns resulting from each of the engineering alternatives evaluated. The results of the sediment transport modeling and structural alternatives analysis are summarized in this report.

The IMP has been developed by Kimley-Horn and Associates, Inc. (KHA) based on modeling assistance and technical input provided by Coastal Technologies Corporation (CTC), a sub-consultant to KHA specializing in coastal processes and IMPs.

Salt Ponds is a tidally-influenced basin located in Hampton, Virginia, approximately 3 miles west of the Chesapeake Bay Bridge Tunnel and 1.7 miles north of Buckroe Beach (Report Exhibits 1a and 1b and Appendix A Exhibits 1 and 2). The basin is connected to the Chesapeake Bay via the Salt Ponds Inlet, which was dredged by the City of Hampton in 1979 to facilitate small boat navigation into and out of the Salt Ponds area thereby facilitating development in the Salt Ponds basin. The City currently maintains the inlet channel at a width of 100 feet throughout its entryway, narrowing to a width of 70 feet where the channel turns southward prior to Red Day Marker #10. The project is currently authorized by environmental permits to a uniform maximum depth of 9 feet below mean low water (MLW). Since the basin was initially dredged in 1979, the area has been developed as a waterfront community with two large marinas and several waterfront neighborhoods. Accordingly, the Salt Ponds is now an active boating center connected directly to the Bay by the Inlet that is utilized by a number of sailboats with deep keels.

At the time the inlet was created, a rock jetty was constructed on the north side of the inlet with the intention of protecting it from shoaling related to near-shore sand transport from the north. The south side of the inlet was initially protected by an existing timber groin that was built in the late 1960s for protection of the adjacent beach. Damage sustained by the timber groin led to its eventual replacement in 2005 by a timber/vinyl sheetpile structure at the same location. Although the structure has been extended 100 feet farther seaward, its elevation is the same as the original groin - approximately 3 feet above MLW throughout its length beyond the high water line. Thus the structure is frequently overtopped during storm events.

Chronic shoaling of the inlet channel has resulted in frequent maintenance dredging, occurring at an average of about every two to three years between 1979 and 2005. More recently, the channel has been dredged almost annually due to shoaling at the mouth as well as in the portion of the channel which turns south into the Salt Ponds. The reported cumulative cost of dredging since June of 2003 has been nearly \$1.1 million, for an average annualized maintenance cost of approximately \$157,000 per year. Much of this cost is associated with mobilizing the private dredging equipment each year. The most recent maintenance dredging event occurred in the first quarter of 2009 at a total cost of $320,000.

The need to dredge has generally been triggered when the bottom depths shoal to less than 6' below mean low water (hereinafter referred to as the "triggering depth"). Accordingly, one of the challenges is to develop engineering measures that increase the time between these triggering events, or secondly reduce the volume of each event. Non-structural measures focus on identifying contracting efficiencies if annualized dredging is still required after installation of engineering measures.

A prior study (URS Consultants, 1992) investigated the hydraulics of the inlet as it affected shoaling. The study's objective was to identify methods for reducing the shoaling rate and the consequent dredging frequency. The 1992 study concluded that the flushing power associated with the volume of water passing through the inlet into the Salt Ponds basin is insufficient to prevent shoaling within the inlet. The study results were not, however, based on a dynamic computational hydraulic model of the Inlet. The study concluded that the inlet was too wide to maintain sufficient tidal velocity to keep sand in suspension and that narrowing the inlet to a width that would do so would render the inlet unsafe for the intended navigation. A reduction in the amount of sand entering the inlet was identified as necessary to reduce the dredging frequency and volume. After evaluating several alternatives, the 1992 study recommended replacing and extending the then-wooden groin on the south side of the inlet with a steel sheetpile groin to a uniform elevation of 8 feet above MLW, dredging the inlet to re-establish the project depth, modifying a reported deposition basin to the north that was intended to act a sand trap, and instituting a program of annual hydrographic surveys in order to more accurately quantify sediment transport processes in the inlet. However, of these recommendations, only the initiation of annual hydrographic surveys, periodic maintenance dredging and the replacement of the original south wooden groin by the current timber/vinyl sheet pile jetty in 2005 were undertaken. No additional engineering measures for protection of the inlet have been initiated.

The Virginia Institute of Marine Science also conducted dye studies in the 1990's designed to determine, among other things, if sand was moving through the north.

As a result of the continued need for frequent maintenance dredging in the Salt Ponds Inlet, in 2008 the City requested that Kimley-Horn and Associates, Inc. (KHA) conduct a review of available historical information related to the project in order to provide a <u>qualitative</u> assessment of measures to reduce sand transport into the inlet. Specifically, the City asked KHA to focus on shoaling patterns as reflected in recent hydrographic surveys, the effectiveness of the southern groin's height, and the northern breakwater's porosity to sand migration. KHA's efforts included a review of the 1992 study of the inlet, examination of dredging records and surveys from the late 1990s through January 2009, analysis of historical aerial photography of the shoreline from Grandview to Buckroe Beach from 1937 through 2007, collection of anecdotal information from current and former City staff and residents of the Salt Ponds, and consultations with local marine contractors.

In addition to the records review, KHA reviewed a written report submitted by an attorney for a group of waterway users (Lawrence Cummings, 2009) containing information about navigation difficulties at the inlet and recommending certain actions to reduce shoaling including increasing the dredging area and raising the elevation of the south groin.

An initial slate of conceptual engineering improvements was developed, consisting of sand tightening the north rock jetty and raising the elevation of the south timber groin. In consideration of costs associated with these improvements (approximately $1.5 million in 2009 dollars) and the lack of reliable quantitative modeling data, the City then requested that KHA model of the inlet to improve the predictive confidence in the suitability of specified engineering alternatives for reducing shoaling in the inlet and as a basis for preparing and Inlet Management Plan.

KHA selected CTC, a firm with recognized expertise in Inlet Management Plans in Florida and other areas to assist in developing a hydrodynamic model to be used in analyzing various engineering alternatives. The model focused on the inlet from the mouth and the immediate offshore area to Red Day Marker #10 in Salt Ponds. Five engineering modifications were selected for evaluation by the inlet model. Following a site visit and consultations with the City, additional alternative modifications were developed and evaluated. The alternatives consisted of the following:

1. Modification A – raise and armor the inside of the existing south jetty along its entire length with the goal of reducing reflected wave energy and sand transport into the inlet.
2. Modification B – raise the south groin to a uniform elevation of approximately 9 feet above MLW by constructing a sand tightened rubble-mound structure over the existing timber/vinyl sheetpile jetty, with the goal of reducing wave energy and sand transport entering the inlet, as well as increasing flushing out of the inlet.
3. Modification C – add a spur or "fish-tail" groin to the existing south jetty with the goal of reducing long-shore current and sand transport into the inlet. If effective, a deposition basin would also be considered during engineering of this modification.
4. Modification D – construct an approximately 100-foot long detached breakwater offshore of the inlet entrance with the goal of reducing wave energy entering the inlet and cross-shore sand transport into the inlet.
5. Modification E – add a spur or "fish-tail" groin to the existing north jetty with the goal of reducing long-shore current and sand transport into the inlet. If effective, a deposition basin would also be considered during engineering of this modification.

Modifications A though E were arranged in a series of alternative configurations designed to determine the relative effectiveness of each modification, alone and in combination with other alternatives, for reduction of shoaling in the Salt Ponds Inlet. Exhibit 3 shows the location and nature of the five modifications selected. Exhibits 4 through 8 show a typical section for each modification.

As part of the modeling effort the consultant team met with the users of the waterway to gain valuable historical knowledge about its operation. KHA and the Hampton Public Works staff conducted a meeting with stakeholders (boaters using the Inlet and other residents in the Salt Ponds) on June 22, 2009 in order to brief stakeholders on the study scope and objectives and to secure information on historic shoaling patterns and navigation difficulties. Comments received from stakeholders following the briefing identified a number of navigation difficulties encountered in the inlet, provided historical information regarding shoaling patters, and recommended a number of measures intended to reduce shoaling. These included raising the elevation of the south jetty, deepening the inlet channel, dredging the "shoulder" of the channel where it turns southward, and placing dredged material farther south on the Salt Ponds public beach when maintenance dredging events are conducted to prevent sand re-suspension and transport back into the inlet.

1.2 Modeling and Alternatives Simulations – Approach and Results

The hydrodynamic simulation component of the Salt Ponds IMP project utilized existing information sources and new data collected specifically for this study. A detailed computer model of the inlet was constructed based on a comprehensive 2009 hydrographic survey of the Salt Ponds basin, inlet, adjacent north and south beaches, and near-shore zone. Sediment samples were obtained from within the inlet and the adjacent north and south beaches in order to infer hydraulic processes from trends in sediment grain size distributions. Tidal data were obtained from the National Oceanic and Atmospheric Administration (NOAA) Sewell's Point station located in Norfolk, Virginia. Local Virginia Institute of Marine Science (VIMS) and NOAA recording stations provided wave and wave/wind data, respectively.

After construction and calibration of the model was completed, historic storm data for Hurricane Isabel (2003) and the February 1993 Nor'easter were used to run simulations of hydrodynamic processes affecting the inlet. These storms were chosen in order to represent the two prevailing wind directions experienced by the area - from the northeast during winter months and from south-southwest during summer and fall months.

Initial hydrodynamic simulations performed with the model indicated that the dominant cause of shoaling within the Salt Ponds Inlet is wave-powered transport of sand from offshore areas toward the inlet (i.e., cross-shore transport). A comparison of the predicted shoaling for the 1993 Nor'easter with the shoaling areas identified by a May 2010 hydrographic survey which occurred about a year after the spring 2009 dredging event found a strong correlation between the predicted and actual shoaling. During this one year period the only major storm event was the November 2009 Nor'easter. Accordingly, any effective engineering approach to reducing sedimentation in the dredged channel will need to address cross-shore sand transport. Furthermore, simulations of the existing jetties indicated that while the north jetty is relatively effective in protecting the inlet from sand borne by northeasterly waves, the south jetty is ineffective against inlet shoaling from sand transported by southeasterly waves.

Following preliminary simulations of inlet hydrodynamics under existing conditions, the five inlet protection modifications selected for analysis were programmed into the model. A suite of simulations (denoted Alternatives #1 - #5) was designed using various combinations of the five modifications and the February 1993 Nor'easter and 2003 Hurricane Isabel storm data previously described. Resulting model predictions of shoaling patterns and wave heights were analyzed in order to determine the relative effectiveness of each modification and combination of modifications for reduction of shoaling. <u>The results indicated that none of the initial five modifications, either alone or in combination, would appreciably reduce shoaling in the inlet from the current condition.</u>

Due to the lack of effectiveness predicted for the initial five modifications, four additional modifications were added to the model. These were as follows:

- Widen the narrowest area or the inlet channel (near Red Day Marker #10) by dredging out a portion of the existing salt marsh on the west side of the channel
- Reconstruct the apparently dilapidated groins located along the public beach south of the inlet
- Extend both the existing north and south jetties to a uniform distance offshore
- Build a large (approximately 300-foot long) breakwater offshore of the inlet entrance in conjunction with raising and armoring the existing south jetty (denoted as Alternative 11)

Simulations including the four additional modifications (denoted as Alternatives #6 - #11) showed that the most effective single modification for reducing shoaling at the Salt Ponds Inlet would be the construction of a large breakwater across the inlet entrance channel in conjunction with raising and armoring the existing south jetty. This modification, denoted as Alternative #11, is anticipated to extend the maintenance dredging cycle from the current interval of 1-2 years to approximately 3-5 years. However the actual dimensions and specific placement of the structure would need to be determined through additional engineering studies and coordinated with the on-going study of the Grandview to Fort Monroe shoreline stabilization study.

1.3 Recommendations and Opinions of Probable Cost

Based on the simulation results of the modifications included in this study and discussions with the waterway users and the City, KHA recommends the following measures be undertaken:

1. Develop plans for raising, armoring and extending the existing south timber/vinyl sheetpile jetty by construction of a rubble-mound structure over the existing jetty. The anticipated cost of constructing the rubble-mound structure over the existing south jetty to a uniform elevation of 9 feet above MLW is approximately $1 million. Although the hydrodynamic model did not indicate a significant change in shoaling patterns would be gained by this structure, it is expected to have a marginally beneficial effect on reducing shoaling in the inlet by preventing periodic overtopping of the structure.
2. Further evaluate the optimal size, location, configuration, and cost of an offshore breakwater at the mouth of the inlet. Additional investigations, including more advanced modeling efforts and preliminary design should be undertaken to gain more information on the most effective location and configuration for the structure as well as the shoaling pattern around such a structure, and its orientation so as not to unreasonably interfere with navigation into and out of the inlet. The northern and southern model boundaries should be extended. This would facilitate an analysis of the efficacy of suitably spaced (both cross-shore and longshore dimensions) offshore breakwaters in stabilizing the shoreline as well as minimizing sedimentation of the inlet. The larger modeling domain will lend to greater degree of confidence in the analysis at local points of investigation. However, any decision regarding the location and size of the structure should be coordinated with the results of a proposed study regarding the stability of the Hampton Shoreline in the Buckroe and Grandview areas in order to more effectively assess system-wide influences and address the concerns of the various Hampton communities affected by coastal processes in a comprehensive manner.
3. The option of narrow-neck widening should not be ruled out if environmental permitting concerns can be effectively addressed. The recommended Alternative 11 (offshore breakwater in conjunction with raising and armoring the existing south jetty) will not reduce the shoaling rate in the narrow-neck area since shoaling hydraulics are different there than at the inlet entrance. Although the shoaling rate at the narrow-neck is lower than at the inlet entrance by an order of magnitude it is still of concern to the users of the Salt Ponds Inlet and could be lessened significantly by widening of the inlet in this area.

The anticipated cost of recommendations #1 and #2 above is approximately $3.0 to 3.5 million in 2010 dollars.

In addition to the structural measures above, the following additional non-structural management measures are recommended to evaluate the effectiveness of any structure undertaken and better document maintenance dredging costs:

1. A dye study should be undertaken to determine if sand migration through the north jetty is a significant source of shoaling within the inlet. If so, the preliminary design of the sand tightening measures previously prepared should be advanced to preliminary design and a cost estimate for the tightening prepared. If undertaken, an examination of the effectiveness of extending the north

jetty and re-orienting the mouth of the Inlet to an east-southeast direction should be undertaken as a means of further protecting the inlet from wave action from the east.

2. The City should install upward looking Acoustic Doppler Current Profilers (ADCPs) twice annually for a 30-day period at the offshore model boundary and near the inlet entrance in order to simultaneously measure water level, current and wave properties. The measurements should be accompanied by pre- and post-deployment bathymetric surveys. These measurements would help analysis during the preliminary engineering stage.
3. The City should continue, and consider expanding the annual hydrographic surveys of the Salt Ponds Inlet.
4. The City should survey and monitor shoreline and inlet cross-sectional profiles at suitable intervals in order to capture seasonal morphological trends. This will facilitate future analysis and modeling efforts and lead to improvements in the structure(s) selected for design and construction. A comprehensive survey and monitoring effort will help explain both seasonal and episodic events and their relative impacts on the performance of the inlet and adjacent shorelines
5. The City should institute a formalized system of record-keeping and survey work to document dredge volumes and beach morphology along the south beach where sediments have been routinely placed during maintenance dredging events.
6. The City should examine whether an On-Call contract for dredging the Salt Ponds and other city waterways or creation of a City dredging capability could help contain or reduce long term dredging costs is advisable.

2. INTRODUCTION

2.1 Existing Conditions and Modeling Background

The Inlet Management Plan is influenced largely by the hydrodynamic model developed to analyze various structural alternatives to reduce the frequency and cost of maintenance dredging the Inlet. The modeling and technical input has been provided by Coastal Technologies Corporation (CTC), a sub-consultant to KHA specializing in coastal processes and inlet management plans (IMPs). CTC used the "Mike21" model developed by DHI, which dynamically couples wave, hydrodynamics, sediment transport and morphology to simulate two-dimensional near-shore processes and optimize jetty configurations at the Salt Ponds Inlet.

The Salt Ponds is a tidally-influenced body of water oriented north/south behind the beaches at Buckroe. It is fed in part by Long Creek, which until 1979 had been blocked from a connection with the Chesapeake Bay by a narrow beach. In 1979, the City of Hampton dredged an inlet to connect Salt Ponds to the Chesapeake Bay at the north end of the Buckroe community in order to gain the economic benefits of creating a boating oriented community ("Salt Ponds Community") in the waterway. The inlet was created at an approximate right angle to the centerline of the Salt Ponds/Long Creek. The north rock jetty, apparently in its present form, was built around 1979 to protect the inlet from the north. The south side of the inlet was initially protected by a timber groin structure that had been built in the late 1960s to protect the beach in the area which later became the inlet opening. Damage sustained by the timber structure led to its replacement with a vinyl sheetpile jetty in 2005.

The Salt Ponds in its present state has an area of approximately 24.7 acres (0.10 $km^{2)}$. The maintained inlet channel design width varies from 70 to 100 feet (21.3 to 30.5 meters) and design depth is -9.0 feet (-2.77 meters) at MLW. The channel is maintained to approximately 75 feet (23 meters) south of Red Day Marker #10 at the northern extent of the Salt Ponds.

Exhibit 1c shows the inlet, the harbor facility and the jetties built to minimize sedimentation of the inlet. The Chesapeake Bay in the vicinity of Salt Ponds Inlet is an extensive shallow area, with the Horseshoe Shoal having typical water depths of less than 18 feet (5.5 m) at MLLW (Fig. 1b).

With a mean-tide range of 2.42 ft (0.74 m) at Sewell's Point, the Chesapeake Bay is characterized by semi-diurnal tide with some daily inequalities in high water (the difference in height between successive high waters). The Bay area in the neighborhood of Salt Ponds inlet is subjected to both local wind-waves, and long-period swells arriving across the Chesapeake Bay from the Atlantic Ocean to the east. The fetch across Chesapeake Bay from the east and north is substantial, varying from 19 miles (30 km) along the east-sector to about 62 miles (100 km) along the north-northeast sector. In addition to the normal seasonal wave climate, the area is also subjected to occasional extreme events – hurricanes and "northeasters" – the most recent of which were Hurricane Isabel in September 2003 and the November 2009 northeast storm created by the remnants of Hurricane Ida.

Abundant wave and tidal information is available for the area. Exhibits 2a-c depict some of the local data collection platforms. The Sewell's Point tide gage is maintained by the National Oceanographic and Atmospheric Administration (NOAA) at the Norfolk Naval Station, while the offshore Chesapeake Bay Light Tower "CHLV2" platform in the Atlantic Ocean is maintained by the NOAA National Data Buoy Center (NDBC). The Thimble Shoal Lighthouse was used by Virginia Institute of Marine Science for wave data collection for a 7-year period between 1988 and 1995.

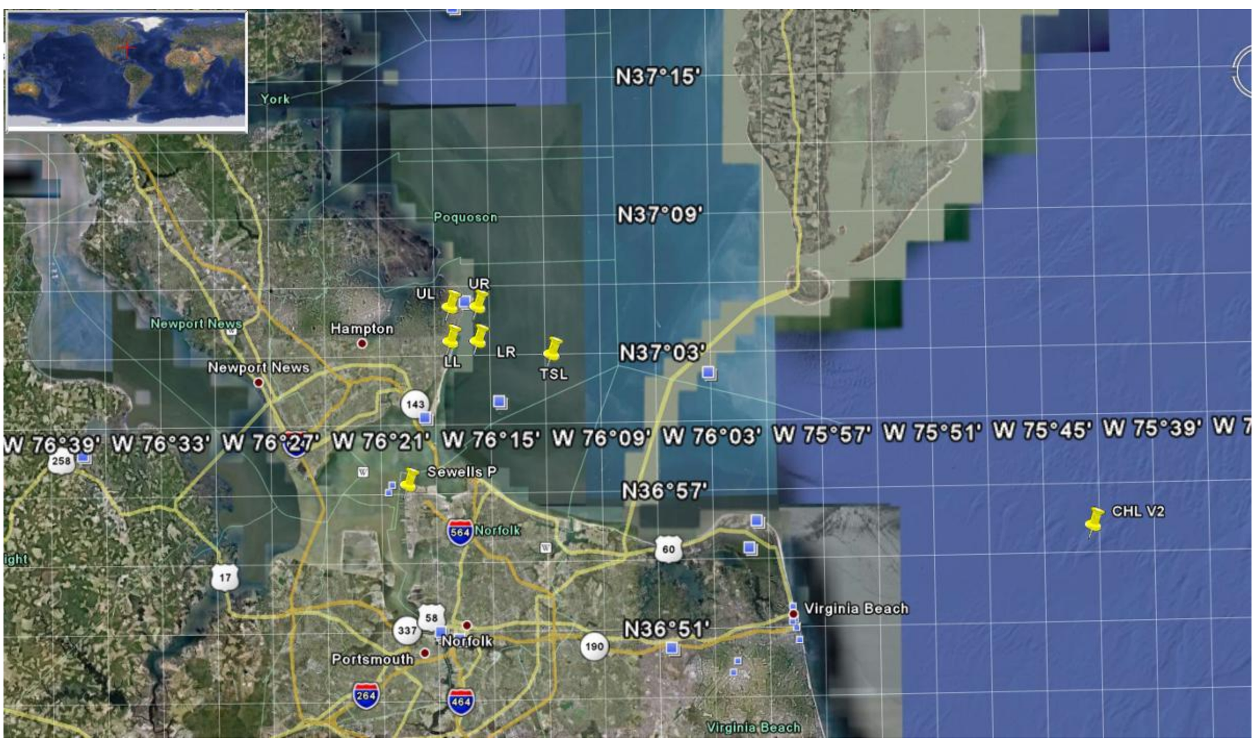


Exhibit 1a: Salt Ponds Inlet location. A Google image of the Salt Ponds location within the Atlantic Ocean - Chesapeake Bay system. Data collection stations: TSL (VIMS, Thimble Shoal Light, NOAA tide gage at Sewell's Point and NOAA/NDBC platform CHLV2. The four corners (UL: upper left; LL: lower left; LR: lower right; UP: upper right) of the model are also shown.

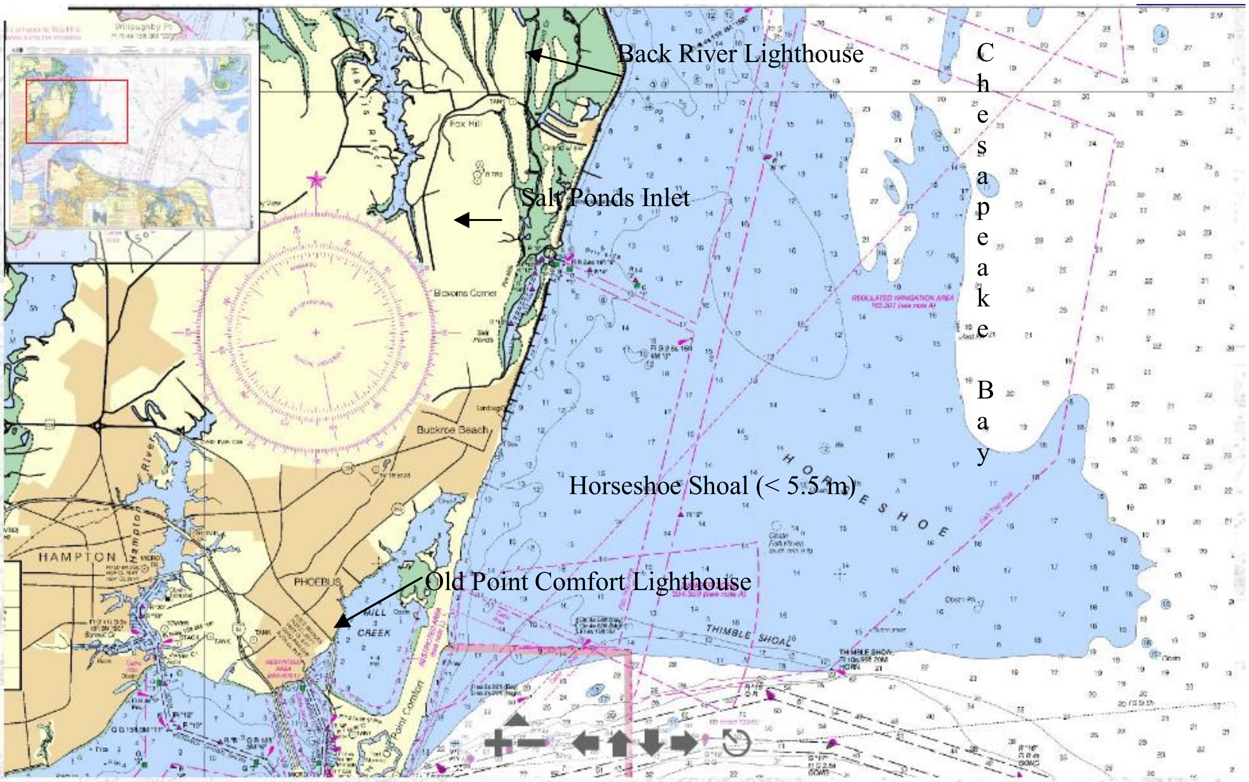


Exhibit 1b. Salt Ponds Inlet and the Horseshoe shoal, Chesapeake Bay.

Exhibit 1c. Google image of Salt Ponds Inlet showing the marina and housing developments.

The Chesapeake Bay shoreline from Old Point Comfort Lighthouse in the south to the former location of the Back River Lighthouse in the north comprises an approximately six mile long sandy beach aligned at an angle of about 10^{o} relative to north. In response to seas and swells, the shoreline is subjected to sand transport both parallel and perpendicular to the shoreline. A groin field is located along the beach south of the inlet, with wooden groins spaced at about 600 feet (180 meters) apart. Many of these appear to be in poor condition and may be ineffective.

Since the connection of Salt Ponds to Chesapeake Bay in 1979, the frequency of dredging required to maintain navigation has increased at a substantial cost to the City of Hampton, which maintains the inlet. Accordingly, the City desires to identify measures to maintain navigation while minimizing recurring maintenance dredging costs.

Exhibit 2a: NOAA Sewell's Point Gage.

Exhibit 2b: NDBC platform CHLV2 in the Atlantic

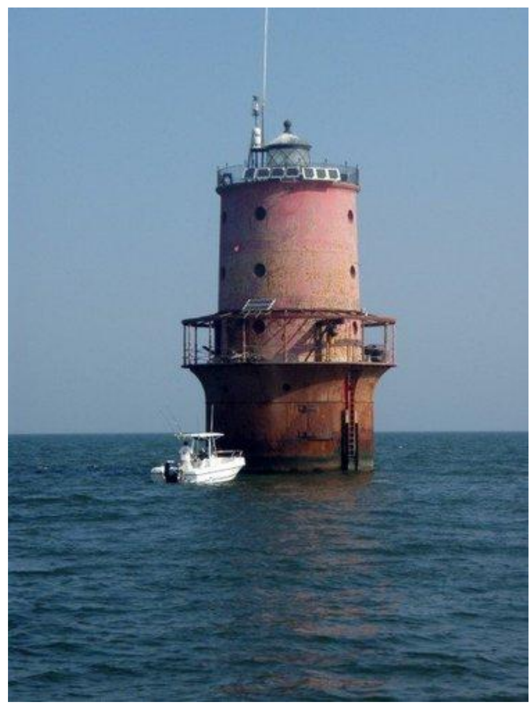


Exhibit 2c: Thimble Shoal Lighthouse – VIMS data collection area

2.2 Inlet Dynamics

Tidal inlets in littoral shorelines typically face continuous shoaling problems. The problem results from the relative influence of the opposing agents of shoaling and flushing. While wave action leads to mobilization of littoral sediments, tidal action is responsible for flushing out the accumulated sediments. The relative influence of wave and tidal power determines the fate of an inlet.

Among the preventative "hard" engineering measures, jetties (shore perpendicular structures at one or both sides of an inlet) are most common. Offshore breakwaters have also been utilized either as a stand-alone measure or in combination with jetties to reduce shoaling. These structures are mostly emergent, but submerged or weir-type structures, often in combination with emergent jetties are also not uncommon. Dredging of shoals and sand by-passing (to minimize jetty-induced down drift erosion) are the two most common curative non-structural measures to alleviate shoaling problems. Most often the solution to chronic inlet shoaling lies in the combination of preventative and curative measures. U.S. Army Corps of Engineers (USACE) manuals published in 1989, 2003 and 2006 discuss the processes and engineering of inlets in detail.

Exhibit 3 contains photographs of the existing jetty configuration of Salt Ponds Inlet. The north jetty is a southeastwardly-curving 575 ft (175 m) long rubble-mound structure. Elevations along the top of the north jetty vary from 5.6 to 9.5 ft (1.7 to 2.9 m) above MLLW. The south jetty is a 410 ft (125 m) long straight wooden and vinyl sheet-pile wall with top elevations varying from 2.3 to 7.5 ft (0.7 to 2.3 m) above MLLW. The current south jetty structure was built in 2005 in order to replace the original 1960s-era timber groin at the same location. The south jetty is likely to face splash or spray overtopping during high water events.

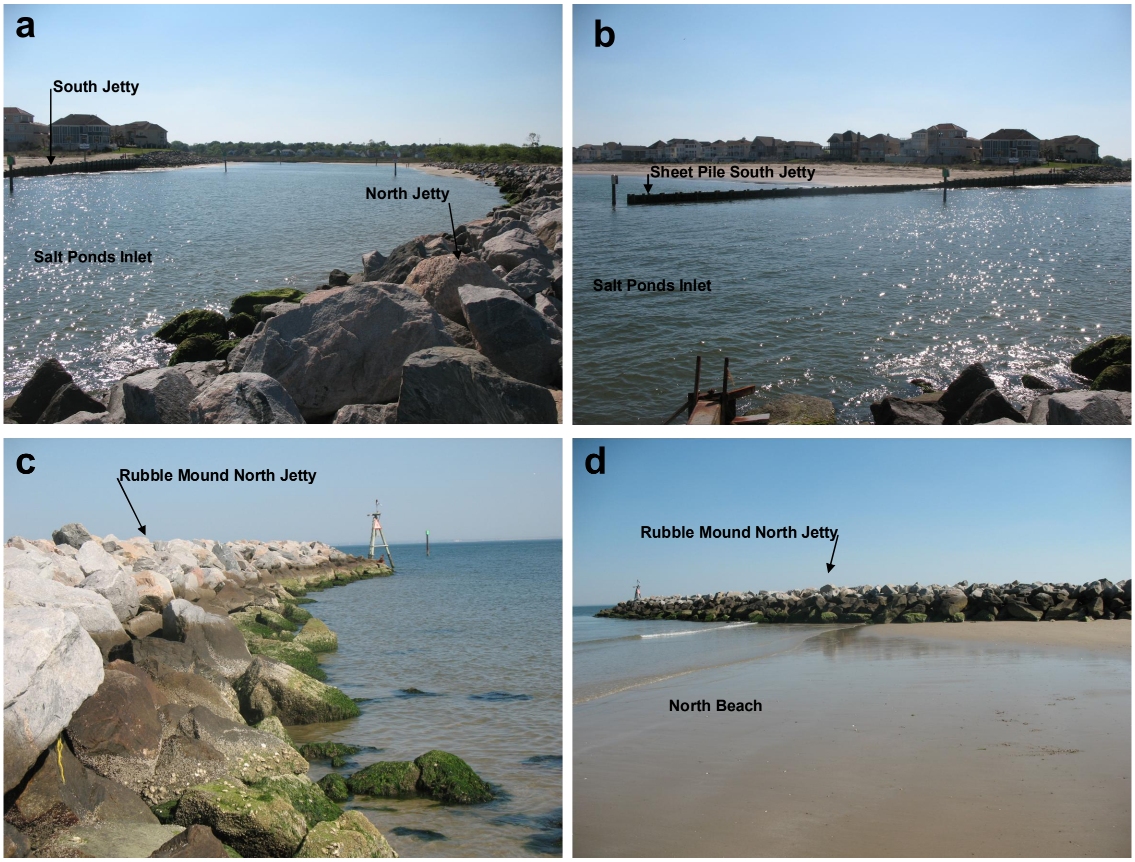


Exhibit 3: Recent Photographs of Salt Ponds Inlet. a. View looking into the inlet. b. View of the sheet pile south jetty. c. View of the rubble-mound north jetty details. d. View of the north jetty from the north.

Exhibit 4 contains dredging rates based on a review of dredging epochs. These rates represent an order of magnitude, partly due to a lack of information to confirm that uniform areas were dredged during these epochs. For the 10-year averaging period from 1979 to 1989 the dredging rate is approximately 8,500 yd^3/year (6,500 m^3/year). The rate is about 13,500 yd^3/year (10,300 m^3/year) for the 6-year period between 2003 and 2009. As the graph shows, there is a shift to a higher rate recently. The dredging event in 2005 may skew the average for this period since the event was precipitated by migration into the inlet of sand placed on the adjacent Buckroe Beach before the south groin was replaced. Another possible explanation for such a shift is that the inlet has been dredged to deeper target depths in recent times. Since there is a shortage of quantitative information about the dredging activities, the use of dredging rates for estimations of shoaling is of limited reliability. Uncertainty associated with dredging volume estimates is discussed further in this report.

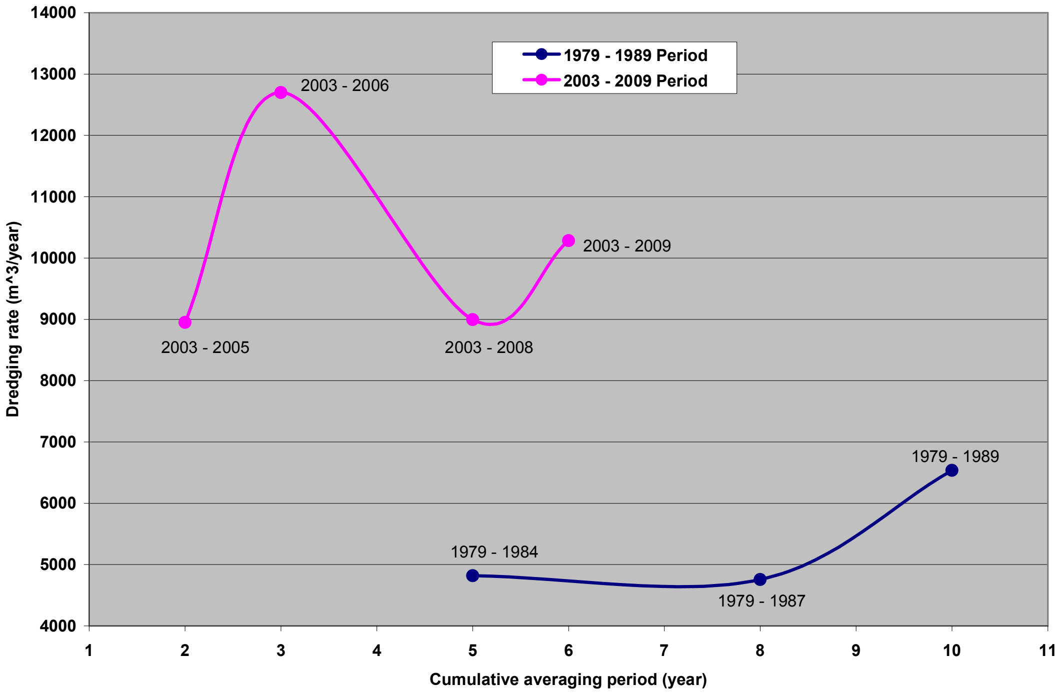


Exhibit 4: Dredging rate inside the Salt Ponds.

3. OBJECTIVES

The objective of the modeling component of this study was to gain a better understanding of the hydraulics of the inlet in order to improve the reliability of recommendations for actions to reduce maintenance dredging frequency. In consultation with the City of Hampton, five alternatives were initially chosen for simulation using the two-dimensional integrated "Mike21" model. Development of the model involved the following steps:

- Definition of the "model domain" and "boundary forcing information" on the basis of desktop analyses of available wave, tide, sediment, morphology and dredging data;
- Identification of relevant model parameters for reasonable results by conducting an analytic assessment of sediment transport and morphologic behavior of the inlet-beach system;
- Validation of the model by qualitative comparisons with available data and by conducting model sensitivity tests;
- Prediction of sediment transport magnitude and direction and morphological changes for each of the four alternatives using a representative storm event scenario.

4. PREVIOUS STUDIES

In response to concerns about maintaining the waterway, a study was conducted by Coastal Planning & Engineering, Inc. (CPE) and URS Consultants in 1992 to gain a better understanding of the inlet dynamics. The 1992 study was primarily based on desktop analyses of available data and dredging records.

This study was followed by a Hampton shoreline monitoring report prepared by URS Consultants (1992). Major relevant findings from the study include the following:

- Since opening of the inlet in 1979, shoaling has been a problem with consequent impacts on navigation. The shoals are being removed by dredging at an interval of every 2 to 3 years.
- At one time the Salt Ponds basin was significantly larger than it is today. A USGS study (USGS, 1986) suggests that the Salt Ponds area has reduced by about 50% – to 2.4 x 10^6 ft^2 (or 222,967 m^2) between 1906 and 1986. The 1906 Salt Ponds surface area was about $5x10^6$ ft^2 (464,515 m^2); the entrance length was about 1550 ft (473 m) and the average entrance channel width was 170 ft (52 m). The present inlet cross-sectional area is about 1350 ft^2 (125 m^2) with non-shoaling area being only 120 ft^2 (11 m^2).
- The estimated maximum inlet shoaling rate was approximately 8,200 cy/yr (6,270 m^3/yr) with about 60% transported from the south and the rest 40% from the north.
- The flood shoal is estimated to grow at a rate of approximately 400 cy/yr (306 m^3/yr).
- The estimated loss of beach sand at a section just north of the Salt Ponds inlet is 2,310 cy/yr (1,766 m^3/yr).
- The tidal prism for the March 1991 conditions was estimated to be about $6.25x10^6$ ft^3 (or 176,980 m^3).
- Maximum velocity in the Salt Ponds inlet is about 0.25 ft/s (7.6 cm/s).
- No sand tightening is required for the north jetty because no sand is passing through the fine rubble core of the of the north jetty.
- Windblown sand over the north jetty accounts for a small percentage of the total transport.

The Virginia Institute of Marine Science also conducted dye studies in the 1990's designed to determine, among other things, if sand was moving through the north.

In 2009 a report was forwarded to the City of Hampton by Larry Cummings entitled "Salt Ponds Waterway – History and Methods to Properly Solve Its Problems" which was submitted on behalf of some of the waterway users. The paper documented navigation difficulties in the Inlet and recommended several solutions including raising and extending the south groin.

5. MEASUREMENTS AND AVAILABLE DATA

5.1 Dedicated Measurements

A bathymetric survey of the inlet and its vicinity and bed-sampling for sediment grain-size distribution were performed in order to provide current, site-specific data for the inlet model. A brief description of the data collection effort used to support the model follows.

5.1.1 Bathymetric Survey

Following maintenance dredging of the inlet in April-May 2009, a bathymetric survey was conducted by East Coast Hydrographic in June 2009 covering the Salt Ponds model domain. Exhibit 5 shows the survey data points in NAD83 Virginia South State Plane Coordinates in meters. The measured depths were reduced to MLLW datum using NOAA Sewell's Point tide data.

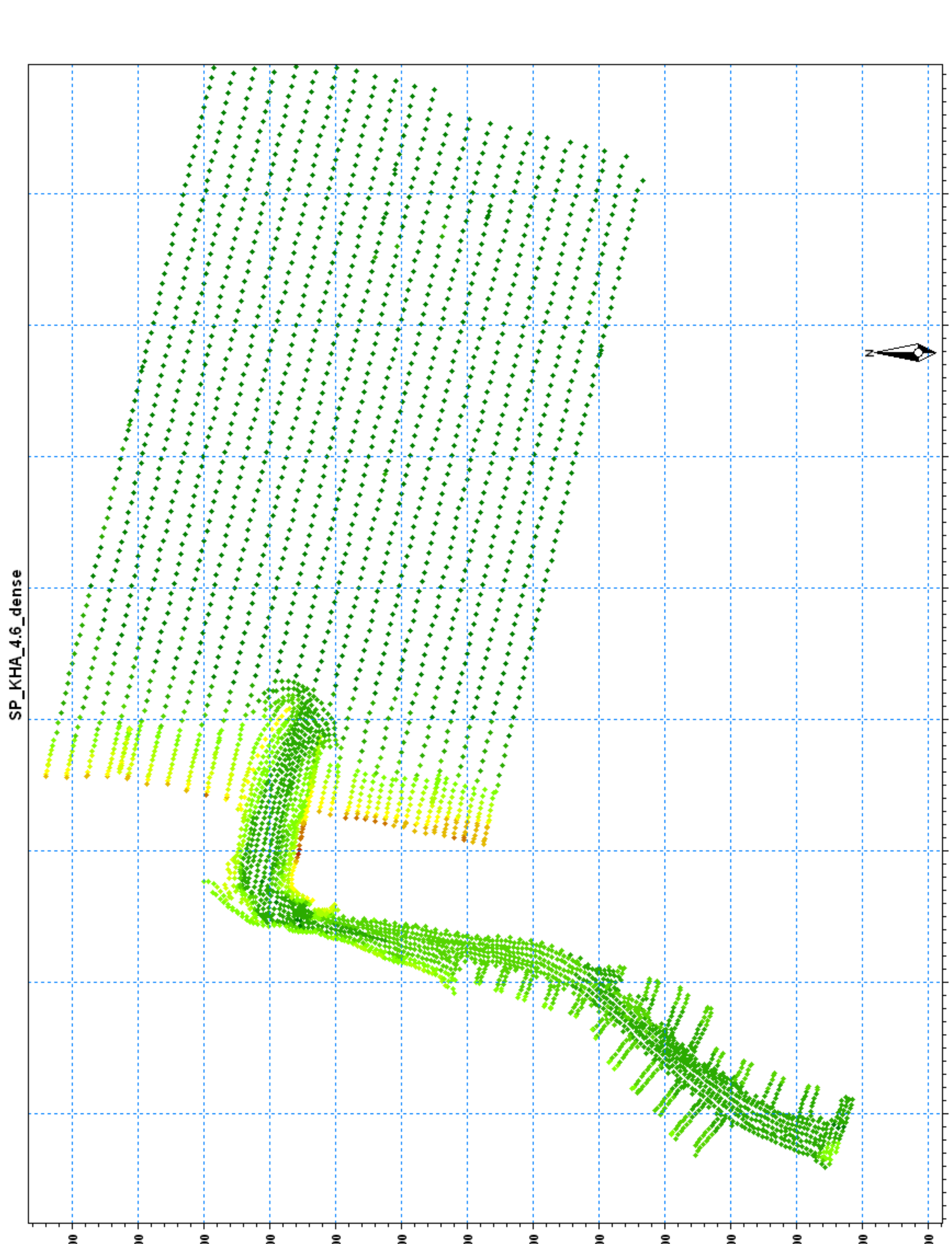
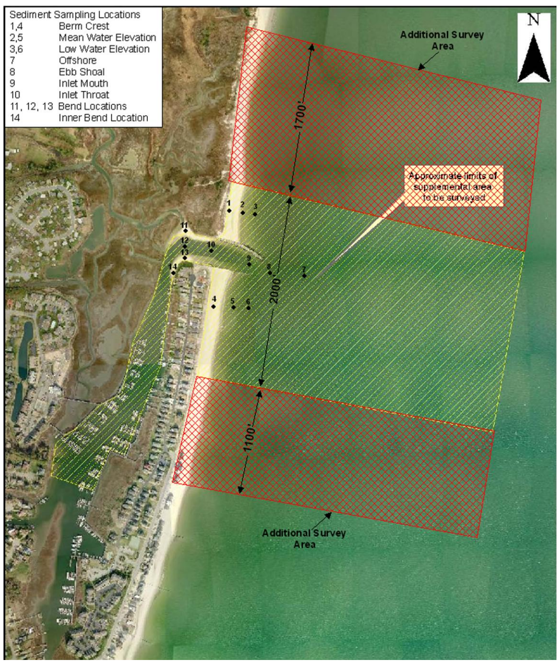


Exhibit 5: June 2009 Bathymetric Survey Data Points (Northing and Easting in meters, Virginia South State Plane Coordinate, NAD83).

5.1.2 Sediment Sampling

A total of 14 samples of surface sediment were collected by Kimley-Horn in July 2009 covering the inlet and its adjacent updrift and downdrift beaches. Exhibit 6 shows the sampling locations and grain-size characteristics in 50th, 84th, 16th percentiles. The sorting coefficient $\sqrt{D_{84} / D_{16}}$ is also shown. Of these data, the inlet sediment sizes were not considered, because they represented the post-dredging condition. Overall, the sediments in the upper dry beaches were coarser than those in the lower wet beaches. There is no recognizable difference or trend in beach sediment sizes between the north and the south sides of the inlet. The sediment data were used to describe grain-size characteristics in the model.



Sediment Data – July 2009

Sample #	Location description	D_{50} (mm)	D_{84} (mm)	D_{16} (mm)	Sorting coefficient (-)
1	Berm crest – north	0.250	0.380	0.140	1.65
2	Mean Water Elevation – north	0.220	0.380	0.110	1.86
3	Low Water Elevation – north	0.180	0.420	0.093	2.13
4	Berm crest – south	0.280	0.390	0.170	1.52
5	Mean Water Elevation – south	0.240	0.380	0.150	1.59
6	Low Water Elevation – south	0.180	0.340	0.092	1.92
7	Offshore of inlet mouth	0.270	0.500	0.170	1.72
8	Ebb shoal	0.130	0.230	0.082	1.68
9	Inlet mouth	0.130	0.230	0.082	1.68
10	Inlet throat	0.110	0.180	0.050	1.90
11	Inlet bend north	0.280	0.380	0.180	1.45
12	Inlet bend center	0.270	0.500	0.170	1.72
13	Inlet bend south	0.370	1.300	0.120	3.29
14	Inlet south of bend	0.300	0.800	0.170	2.17

Exhibit 6: Sediment Sampling Locations and Analyzed Data.

5.2 Data from Secondary Sources

Several attempts to secure on site tide wave and storm data were unsuccessful due to equipment failure. Accordingly, tide, wave and storm data were primarily derived from secondary sources. The locations of these data collection platforms (Exhibit 2) were not located near the model boundary. As the model was used as an optimization tool to indicate the relative performance difference among the simulated structural alternatives, the use of non-local data was in accordance with accepted industry practices.

5.2.1 Tide at Sewell's Point

Exhibits 1a and 2a show the location and a photograph of the data collection platform of NOAA tidal station Sewell's Point, respectively. The geographical location of the station is 36°56.8'N; 76°19.8'W. The tide is semi-diurnal with some degree of inequality in high water, as reflected in the amplitude form number of 0.21.

Table 1: Tidal elevations at Sewell's Point, Hampton Roads, VA.

Datum	**feet**	**m**
Mean Lower Low Water, MLLW	0	0
Mean Low water, MLW	0.13	0.0396 ~ **0.04**
Mean Sea Level, MSL	1.35	0.4116 ~ **0.41**
Mean Tide Level, MTL	1.34	0.4085 ~ **0.41**
Diurnal Tide Level, DTL	1.38	0.4207 ~ **0.42**
Mean High Water, MHW	2.56	0.7805 ~ **0.78**
Mean Higher High Water, MHHW	2.76	0.8415 ~ **0.84**
North American Vertical Datum, NAVD88	1.65	0.5031 ~ **0.50**
Highest recorded	8.02	2.4451 ~ **2.45**

Established tidal elevations and datums are shown in Table 1. The mean tidal range at the station is 2.43 ft (0.74 m), and the highest recorded level of 8.02 ft (2.45 m) above MLLW is slightly higher than which was measured during Hurricane Isabel (7.9 ft (2.4 m) above MLLW; discussed in Section 4.2.4).

5.2.2 Waves at Thimble Shoal

Directional wave data used in the model was obtained from data collected by the Virginia Institute of Marine Science in cooperation with the Virginia Department of Conservation and Recreation for the period 1988-1995. Data collected at the station located 2.5 nautical miles northeast of the Thimble Shoal Light (TSL) in lower Chesapeake Bay (Exhibit 1a and 2c) was used due to proximity to the Salt Ponds project area. It was located at 37°2.4'N, 76°11.9'W; approximately 4.9 miles (7.9 km) east of the Salt Ponds Inlet entrance, at a depth of about 18 ft (5.5 m). The data is available on-line at http://web.vims.edu/physical/research/VIMSWAVE/VIMSWAVE.htm.

Waves were measured and recorded using self-contained Sea Data Model 635-9RS directional wave gages with Paro Scientific high precision quartz pressure transducer, KVH digital compass, and Marsh-McBirney 2-axis electromagnetic flow sensor. The gages were attached to weighted tripods mounted on the bay floor and retrieved at monthly intervals for data recovery and servicing.

During the measurement period of about 8 years, no major storm occurred. However, several small extra-tropical storms were reported. These storms are common every year. Among them, a 3.5-day long February 1993 storm has been-used by other investigators (e.g. Lin et al). Exhibit 7 shows the wave height (Hm0) of less than 1 ft (0.3 m) and water-level during this storm, as well as for the storm in April 1991. The February 1993 event was used to examine model performance and for optimization analysis.

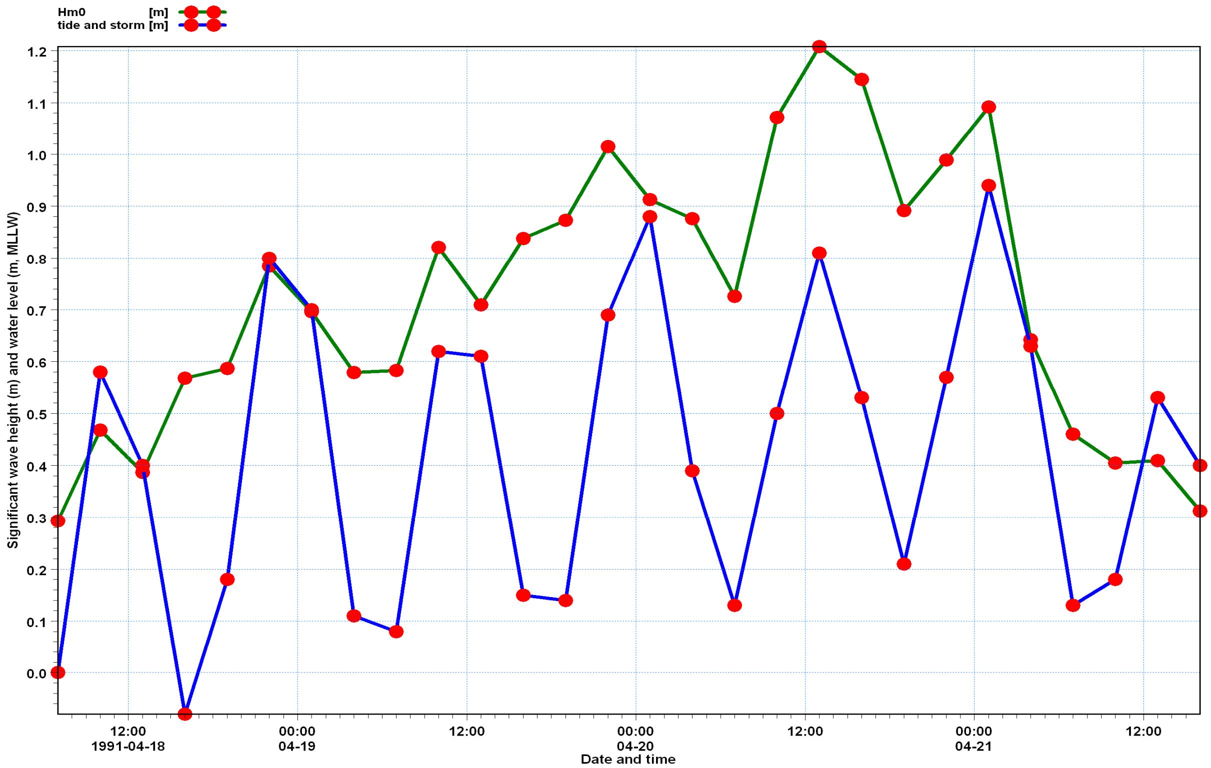


Exhibit 7a: April 1991 Storm Wave Height and Water Level.

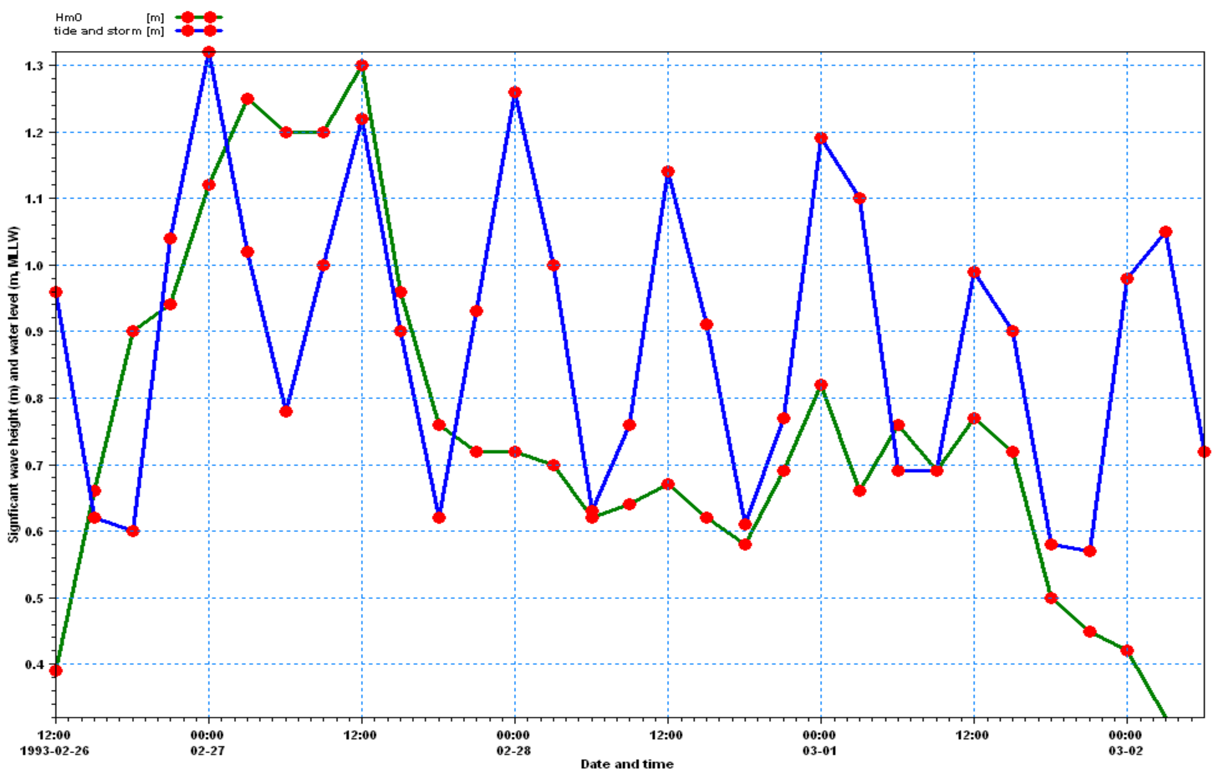
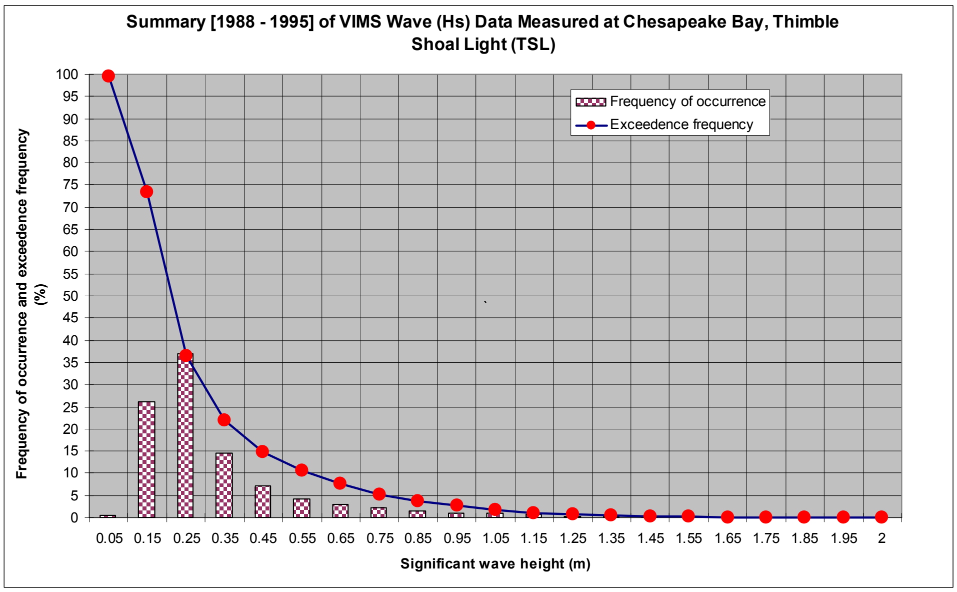


Exhibit 7b: February 1993 Storm Wave Height and Water Level.

A frequency analysis of the measured wave height and period is presented in Exhibit 8. During the measurement period of about 8 years, no cyclonic storm occurred. The 50th percentile wave height is only about 0.66 ft (0.2 m). The wave height occurring for 12 hours in a year and representing an exceedence frequency of 0.14% is 5.25 ft (1.6 m). Accordingly, this wave height was used for estimating the depth of closure for sand transport in the nearshore zone (Hallermeier, 1978, 1981).

The wave periods show a bi-modal distribution indicating the influence of both wind waves and swells. This distribution is well-known for the lower Chesapeake Bay with propagation of swells from the Atlantic. The modal wind-wave period of 5 seconds (s) is associated with an exceedence frequency of 85%, while the swell modal period of 9 s is associated with an exceedence frequency of 42%.



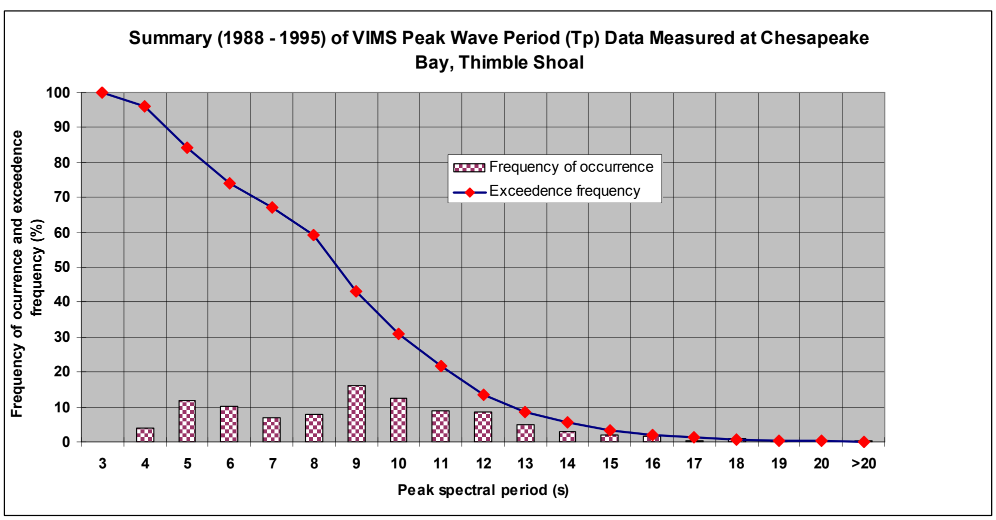


Exhibit 8: Frequency of Occurrence and Exceedence Frequency Wave Heights and Periods Measured at TSL for the 1988-1995 Period.

5.2.3 Winds and Waves at NDBC station CHLV2

The NOAA station CHLV2 (Chesapeake Bay Light, Virginia) is located offshore in the Atlantic Ocean (Exhibits 1 and 2). The station records meteorological, wave and environmental parameters at every hour at sea level. Exhibit 9 shows the wind rose for the period 1988-94. Wind speeds are adjusted to represent an elevation 33 ft (10 m) above mean sea level. The major wind direction is aligned along northeast-southwest axis. Coincidentally, the lower Chesapeake Bay western shoreline from Buckroe to Grandview is aligned in a similar fashion (the shoreline alignment is 10^{o}N). The wind rose indicates the predominance of bi-directional wave climate and sand transport along the shoreline in the vicinity of the Salt Ponds Inlet.

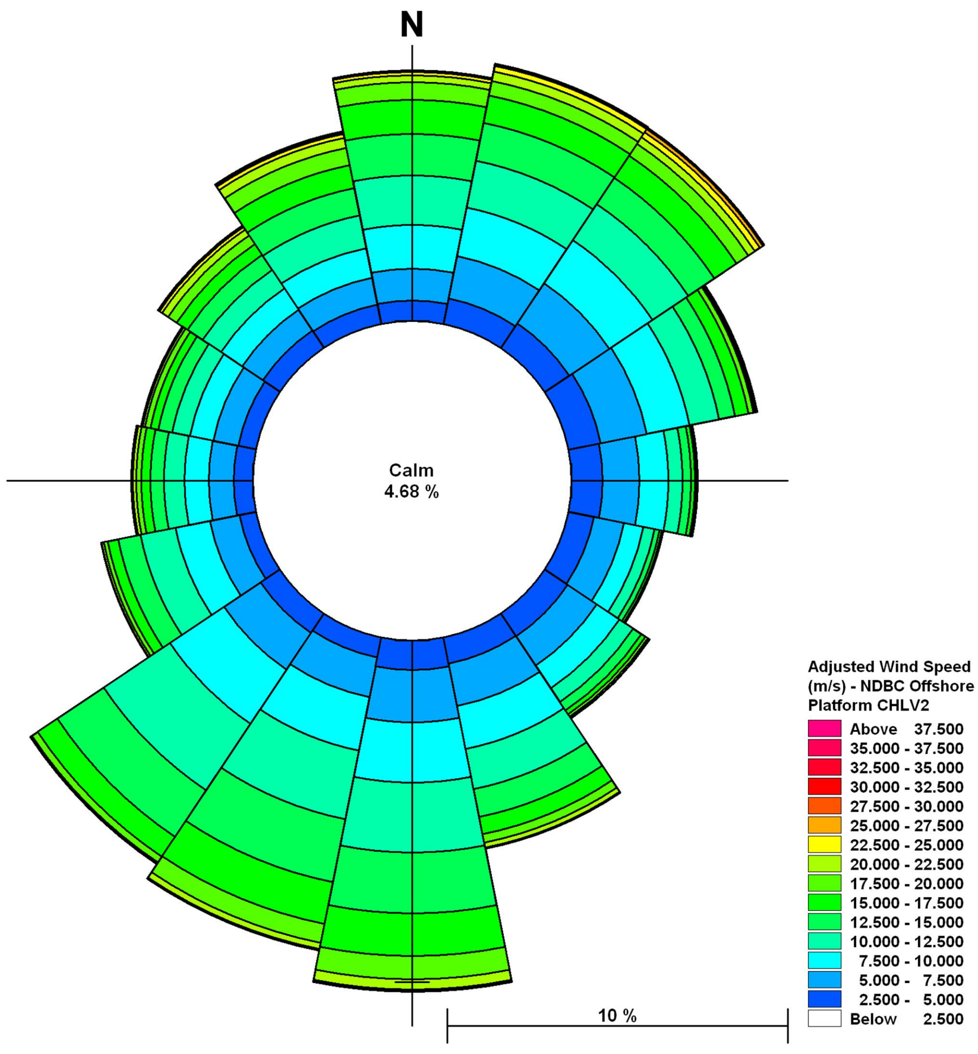


Exhibit 9: Adjusted wind speed rose based on CHLV2 data for the 1988-94 period.

5.2.4 Storm Data

In addition to small extra-tropical storms, the Chesapeake Bay is frequented by cyclonic storms. Bretschneider (1959) documented four hurricanes in the last century that affected Chesapeake Bay. Among these, 3 hurricanes (Hazel, Connie and Dianne) occurred during the 1954 – 55 period, with two of them (Connie and Dianne) occurring in August of 1955. A number of additional tropical cyclone have impacted the region since, including Hurricane Floyd in September of 1999 and Hurricane Isabel in September of 2003. Such storms have high morphological significance for Chesapeake Bay shorelines. Exhibit 10 shows some storm parameters for the area. Analysis of all Category 2 (96 – 110 mph) storms by NOAA shows that the striking frequency of this storm at the Chesapeake Bay mouth is 43 years for data through 1999 (Fig 10a). For other storm categories storms, return periods are shown in Exhibit 10b. The most recent hurricane to move through the Salt Ponds vicinity was Hurricane Isabel, which struck the area as a marginal Category 2 storm in September 2003. In absence of other relevant information, this storm was used for engineering analysis and for comparison with the February 1993 local storm. Exhibit 10c shows the measured tide and associated storm surge measured at Sewell's Point. The peak surge for Hurricane Isabel reached 7.9 ft (2.4 m) above Mean Lower Low Water or an elevation of 6.23 ft (1.90 m) above NAVD88 at 5 pm on September 18, 2003. Exhibit 10d shows the storm surge together with wave heights measured at CHLV2.

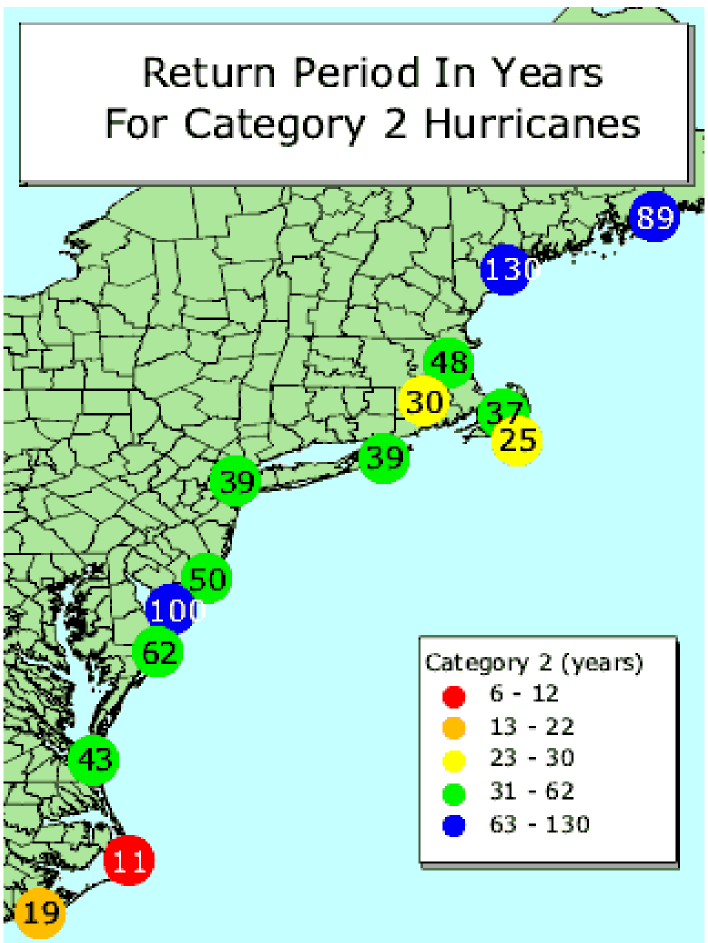


Exhibit 10a: Return period of Category 2 Hurricanes on northern Atlantic US coast

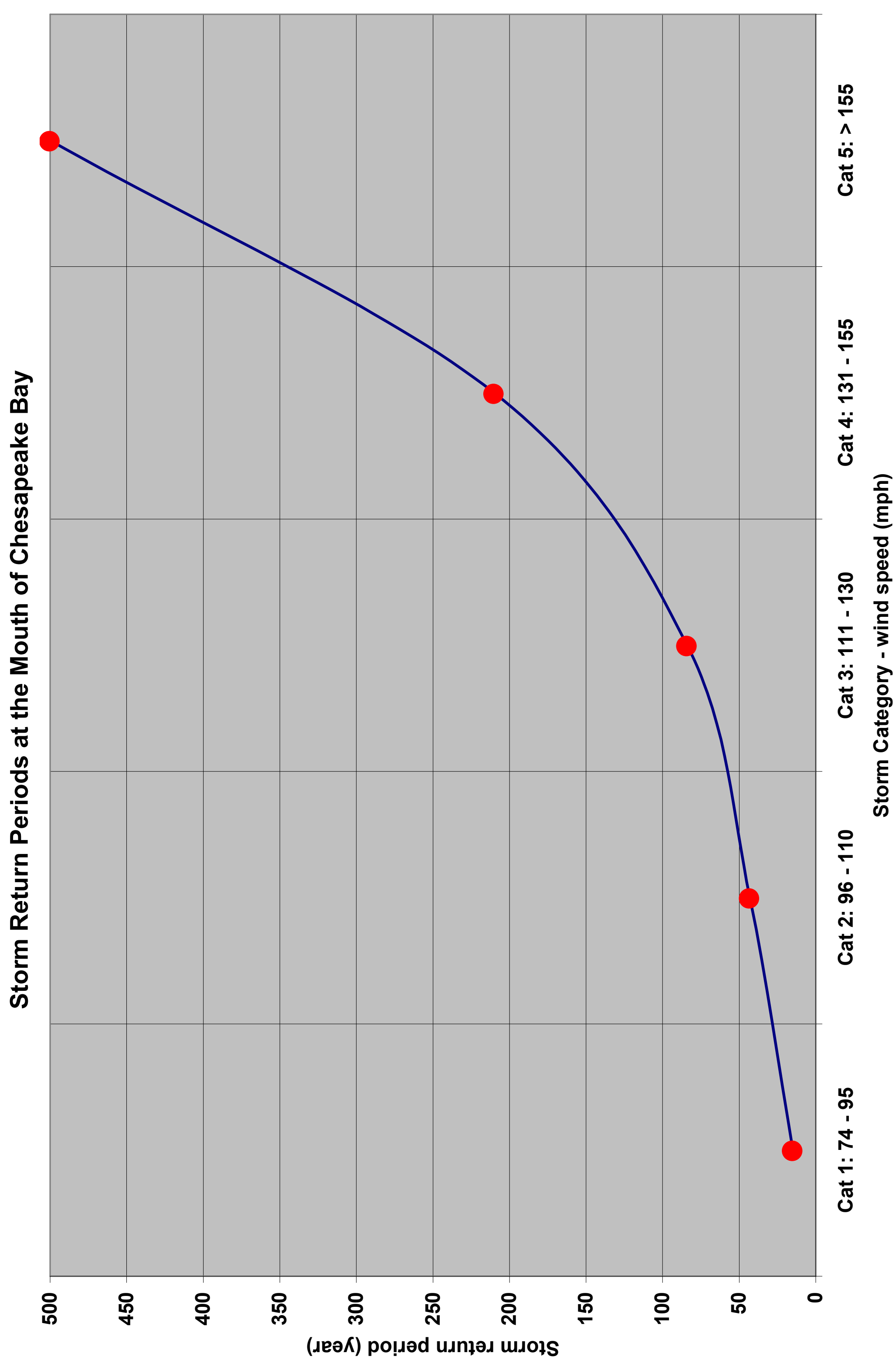


Exhibit 10b: Return periods of different category storms for Chesapeake Bay.

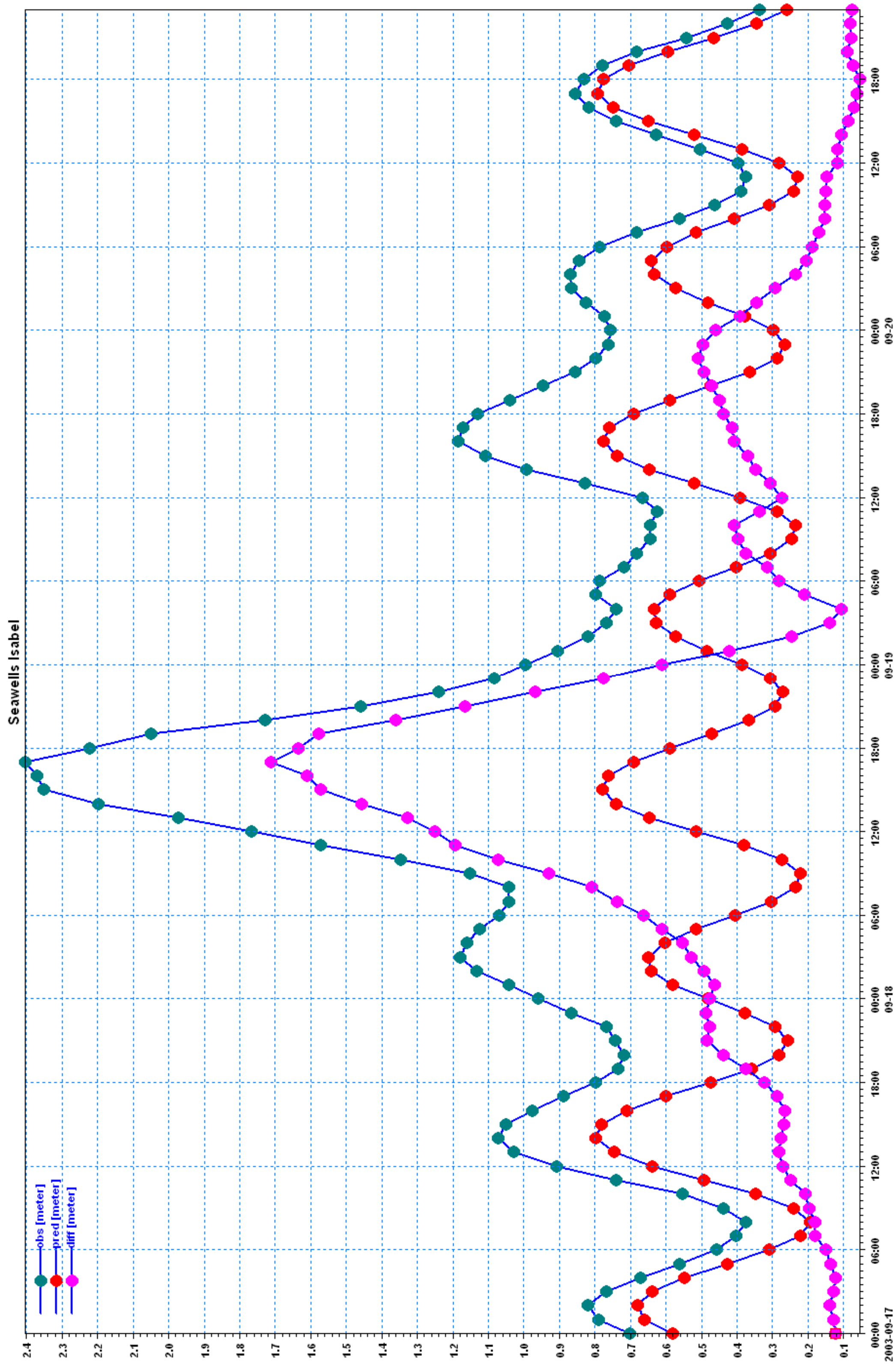


Exhibit 10c: Tide and storm surge measured at Sewell's Point during Hurricane Isabel.

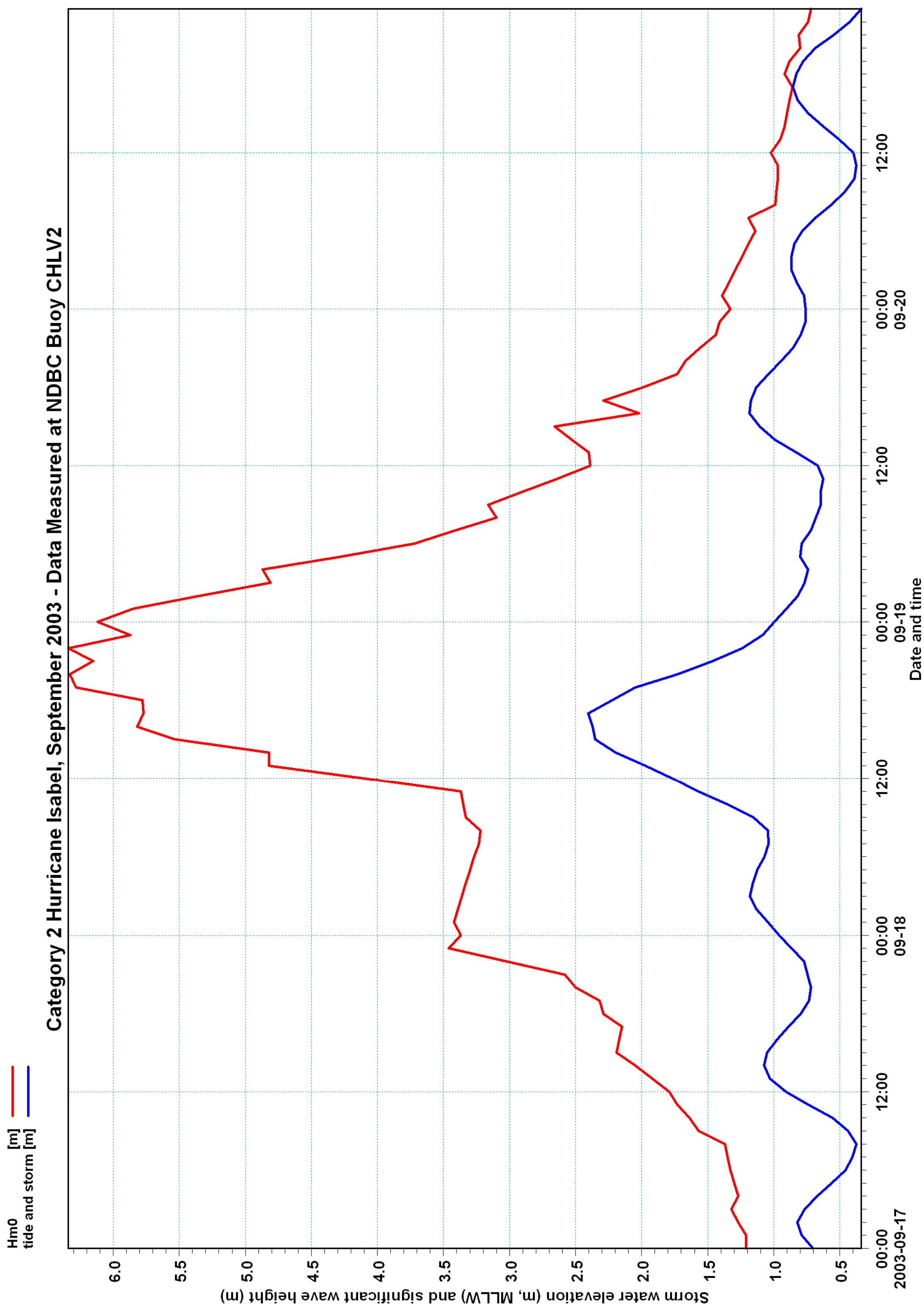


Exhibit 10d: Water level and significant wave height measured at CHLV2.

6. INTEGRATED MODEL

6.1 Modeling Rationale

An integrated computational modeling approach dynamically coupling spectral wave, hydrodynamics, sediment transport and morphology was adopted for this study. The Mike21 two-dimensional, flexible mesh modeling suite was applied for the purpose (DHI, 2009a, 2009b and 2009c). A two-dimensional dynamically coupled model considers the variables interactively and simultaneously in shallow water processes.

In absence of site-specific tide data, NOAA Sewell's Point water levels were used in the model. The model was used as a tool for engineering analysis in order to screen and optimize structural alternatives selected for inclusion in the study. This approach, which is supported by Vreugdenhil (2006), allowed a focus on the relative advantages and disadvantages of the simulated alternatives since boundary conditions and parameters remain unchanged.

As noted above, in absence of more site-specific data, the February 1993 local storm was chosen for qualitative calibration of the model as well as for optimization analysis, while the September 2003 Hurricane Isabel and the April 1991 storm were used for comparisons with the outputs of February 1993 storm. These comparisons provided a means of improving confidence in model performance.

All simulations were made in fully spectral in stationary mode. A moveable boundary was applied to ensure flooding and drying of the shoreline by water level changes.

6.2 Model Set-up

The model domain (study area and boundaries) encompassed the Salt Ponds area and the Chesapeake Bay area off the inlet mouth. The updrift and downdrift boundaries were located about 1,000 ft (300 m) off the inlet mouth, and the offshore open boundary was located at 3,300 ft (1,000 m) off the shoreline.

Exhibit 11 shows the model domain and the triangular computational elements. The whole domain was divided into two sub-domains: the coarse-grid offshore region and the fine-grid inlet and the nearshore region. The total numbers of computational nodes and elements were 2,385 and 4,248, respectively.

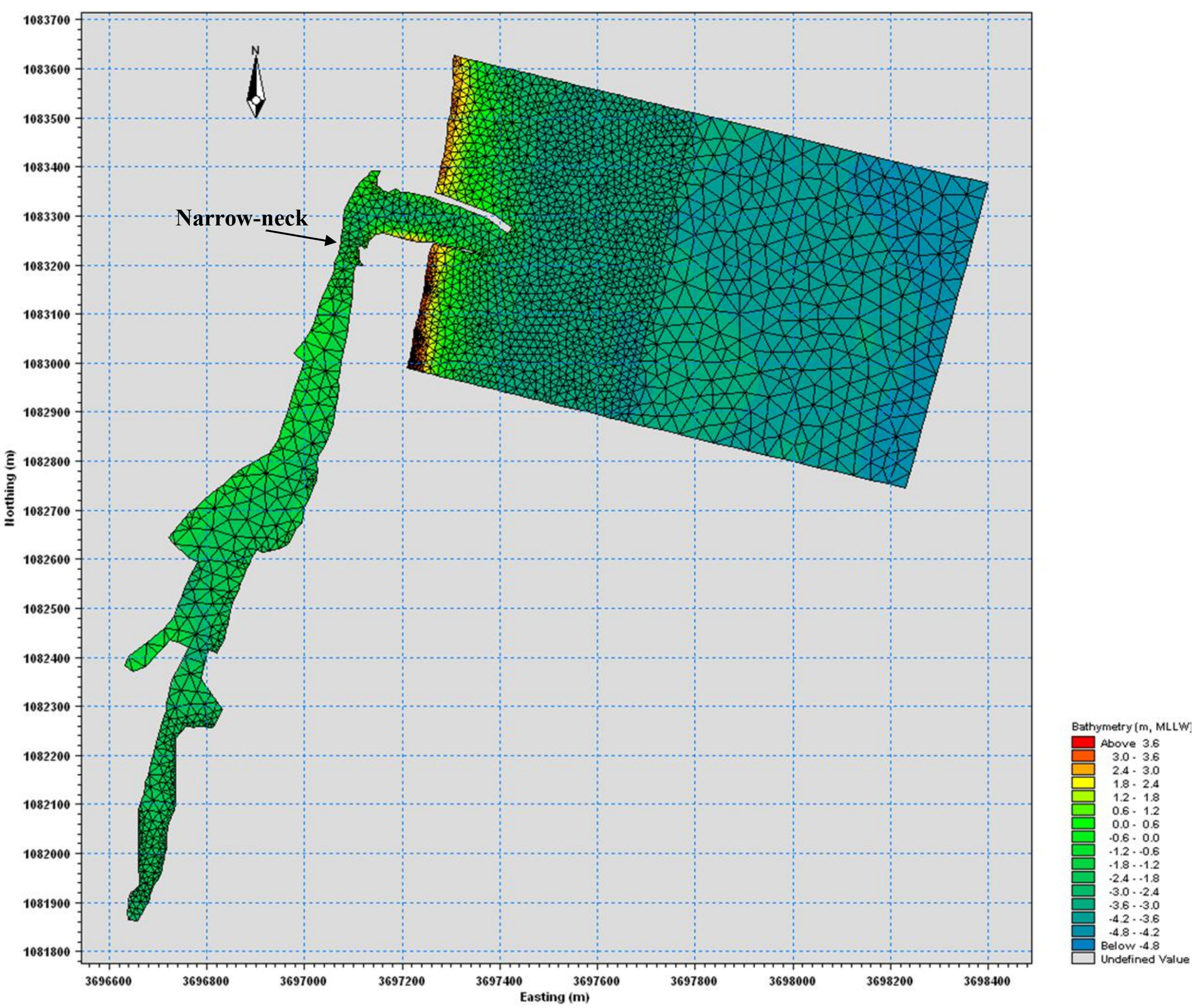


Exhibit 11: Model Computational Domain.

The model had three open boundaries – north, south and offshore. The Long Creek continuation of the Salt Ponds both at north and south ends was assumed to be closed.

Among the sediment data discussed in Section 4.1.3 and Exhibit 6, the samples at low and mean water elevations, north and south of the inlets are considered representative for model application. Average median diameter of these samples is 0.205 mm and the sorting coefficient is 1.88. Both the jetties were assumed as solid and impermeable structures.

Exhibit 12 shows the model bathymetry for the area of interest representing the post dredging condition in May 2009.

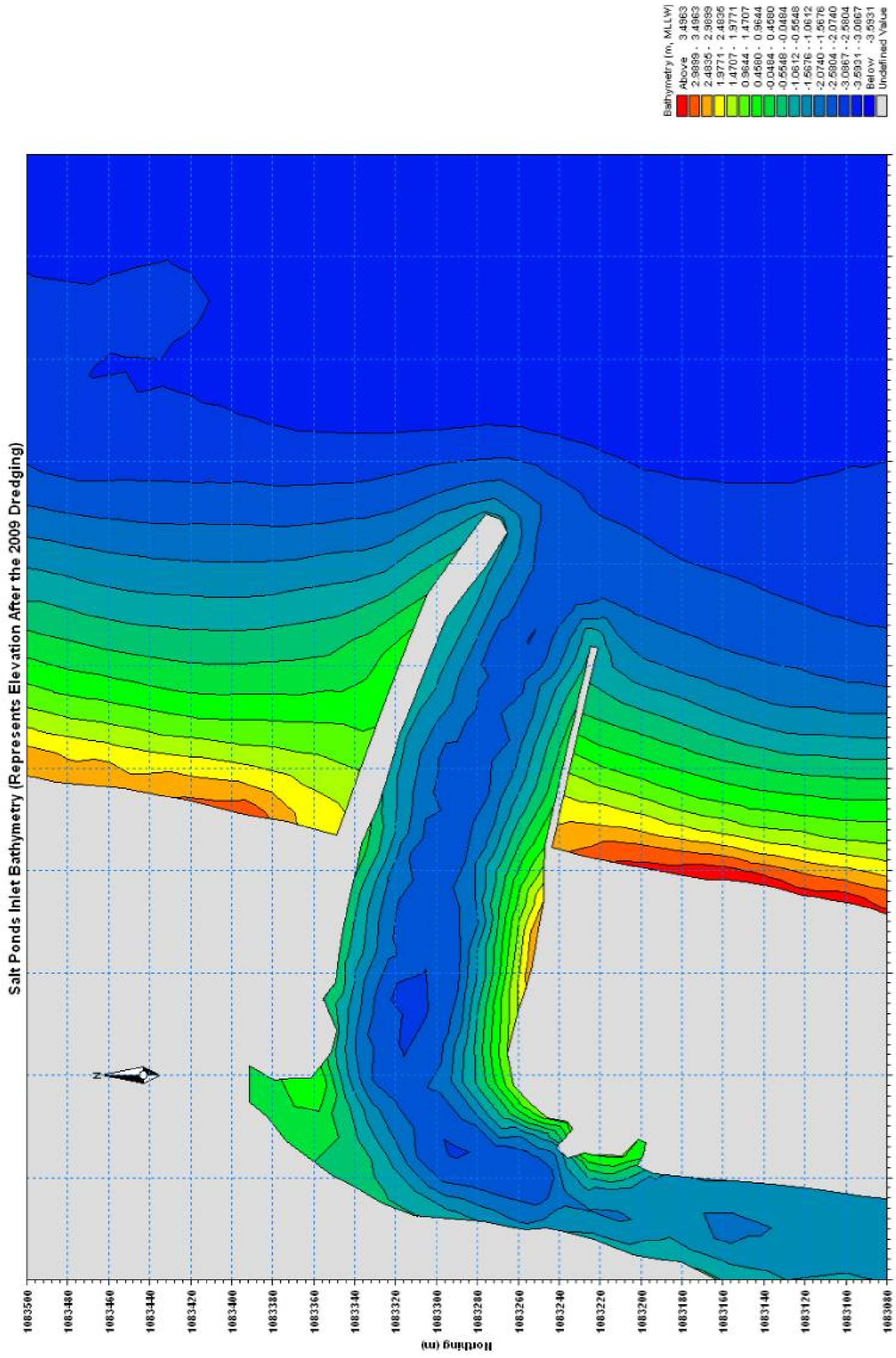


Exhibit 12: Salt Ponds Inlet Bathymetry

6.3 Boundary Forcing

A model simulates conditions (such as water level, current, wave and sediment transport changes in space and time) that are applied or "forced" upon it at the open boundaries. In the absence of site-specific data, forcing of these parameters is performed using data derived from different sources representing different locations. This is the best practical approach, but not ideal for model calibration. However, as the overall objective of the model used in this study was to optimize alternatives, the relative differences between the simulated alternatives were of primary significance.

Waves and water levels were forced at the north, south and offshore open boundaries for events of interest. Three boundary forcing scenarios were considered: the February 1993 northeasterly storm, the April 1991 northeasterly storm, and the 2003 Category 2 Hurricane Isabel.

6.4 Model Performance, Calibration and Sensitivity Analysis

As mentioned earlier, there were no measurements dedicated for model calibration. Instead, model performance was examined by comparing the simulated results with the available information from the CPE and URS (1992) report. Note that this report provided the only published information available for comparison with the model results. Model results were validated by internal checking and sensitivity analysis. It should also be noted that model analysis of processes and optimization of alternatives are not comparable to desk-top studies. Desk-top studies are mostly qualitative and cannot quantitatively predict a situation of interest in space and time or optimize alternatives. A brief description follows.

6.4.1 Current

In order to estimate current velocities in the inlet and compare the estimated values with the reported maximum current data, the February 1993 northeasterly storm was applied at the model boundary. According to the CPE and URS (1992) report, maximum current inside the inlet is about 0.26 ft/s (0.08 m/s). This is an order of magnitude estimate because the report did not specify the location and time of the reported current. Exhibit 13 shows the maximum wave and tide-induced depth-averaged currents during the falling tidal phase. During the February 1993 storm simulation, the current magnitude varied from 0 ft/s (0 m/s) in very sheltered and shallow regions to 0.40 ft/s (0.12 m/s) near the inlet mouth and at the narrowest part of the inlet. The maximum current at the point where the inlet turns southward toward the Salt Ponds basin was approximately 0.20 ft/s (0.06 m/s) (Exhibit 13a). The maximum current inside the inlet could reach up to 1.64 ft/s (0.5 m/s) during severe storms such as Hurricane Isabel (Exhibit 13b).

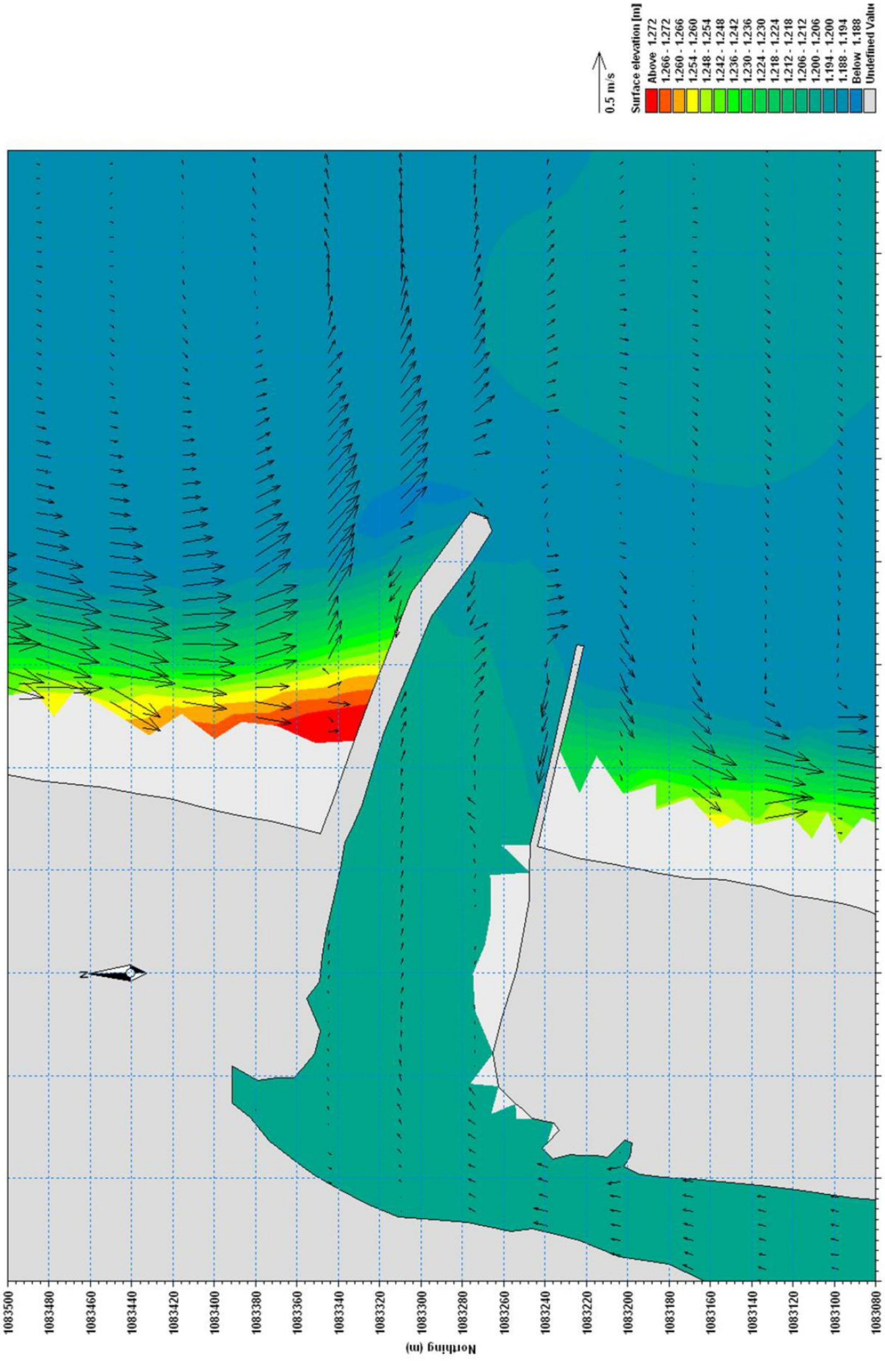


Exhibit 13a: Snapshot of maximum falling tide depth-averaged currents during the February 1993 storm.

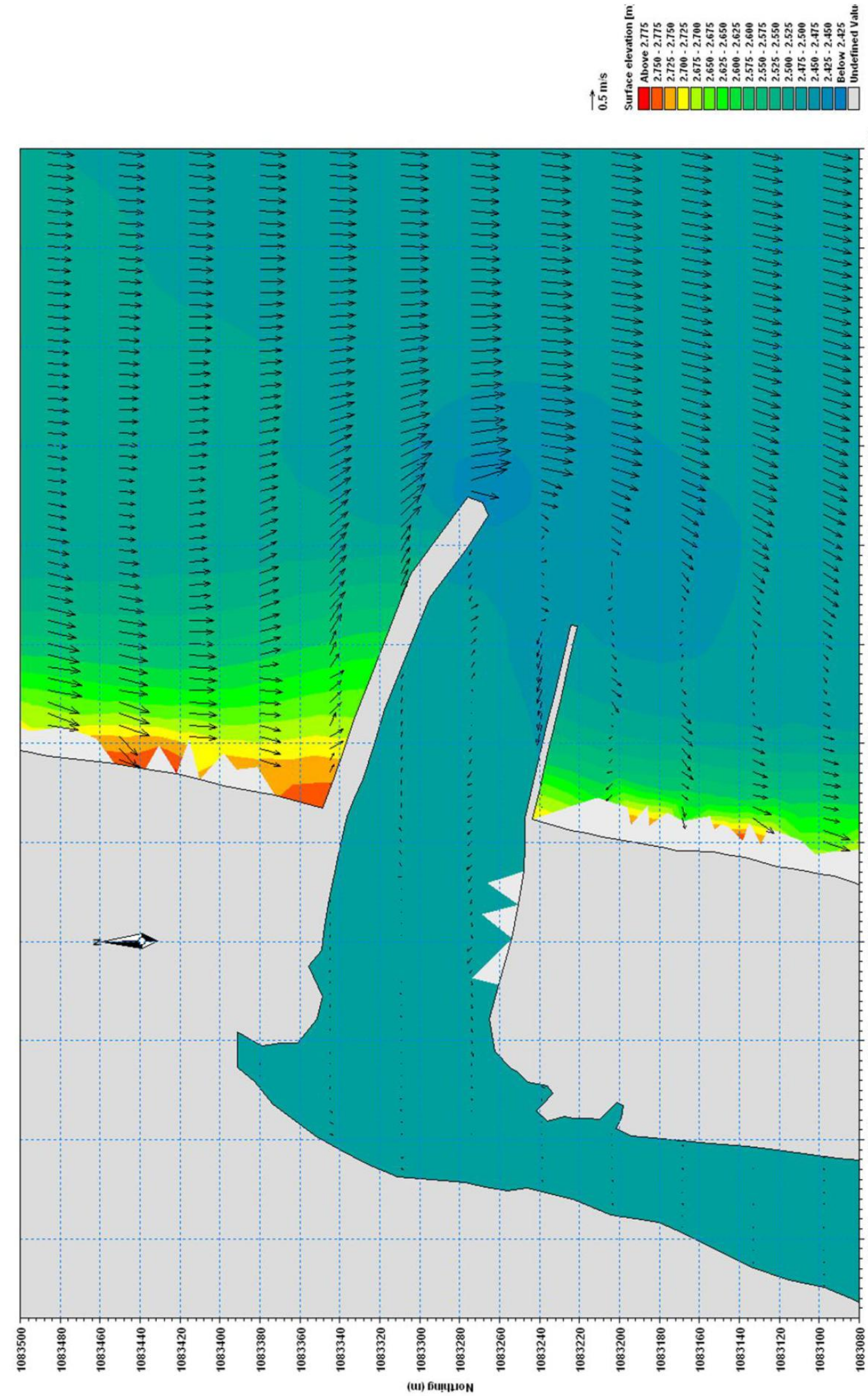


Exhibit 13b: Snapshot of maximum falling tide depth-averaged currents during the peak surge, Hurricane Isabel.

6.4.2 Tidal Prism

According to the CPE and URS (1992) report, the tidal prism for the March 1991 conditions was estimated at 231,481 yd^3 (176,980.3 m^3). This estimate was presumably computed as the product of tidal range and inlet surface area, although the report did not indicate the tidal conditions used in arriving at this estimate. Model simulation showed that during the February 1993 storm the flux-based tidal prism estimated at the inlet entrance varied from 161,463 yd^3 (123,447 m^3) to 330,404 yd^3 (252,612 m^3).

6.4.3 Longshore Transport

Model simulations provided an opportunity to estimate the amount of sediment moving parallel to the shoreline (longshore sediment transport) for the simulation period. For the 3.5-day February 1993 storm, net longshore sediment transport across the sections shown in Exhibit 19 varied from 9.2 to 60.1 yd^3 (7 to 46 m^3) in sections south of the inlet (LL' and MM'), and from 18.3 to 52.3 yd^3 (14 to 40 m^3) in sections north of the inlet (JJ' and KK'). A comparison of this transport volume to annual transport rates would require reliance on considerable uncertainties and simplified assumptions. Therefore, no attempt has been made to provide such an estimate. The net transports can be compared with the reported shoreline volume loss rate of 2,300 yd^3/year (1,766 m^3/year) at a section north of the inlet (CPE and URS report, 1992).

6.4.4 Inlet Shoaling Rate

The 3.5-day February 1993 storm simulation produced a net shoaling amount in the area from the entrance to the narrow-neck located south of the inlet's 90-degree bend (Exhibit 18a and 18b and Table 5) of approximately 314 yd^3 (240 m^3). For the sake of comparison, this volume, if extrapolated, would represent an annual shoaling rate of about 32,737 yd^3/year (25,029 m^3/year), assuming that the February 1993 like storm condition prevailed throughout the year. Available historic bathymetric surveys were used by the consulting team to estimate shoaling and erosion volumes in the area shown in Exhibit 19. As shown in Table 2, the net shoaling rate estimated from the historic surveys varied from about 5,100 yd^3/year (4,000 m^3/year) during 2003-04 to about 26,000 yd^3/year (20,000 m^3/year) during the following period in 2004-05. The significantly lower net shoaling rate during the 2003-04 period is anticipated to have been caused by Hurricane Isabel in September 2003. As demonstrated in Section 5.6 of this report, scour occurring along boundaries during hurricane-scale events overwhelm sedimentation inside the inlet, giving the impression of net loss.

Table 2: Volume changes estimated from historic bathymetric surveys.

Period	Shoaling rate		Erosion rate		Net	
	(m^3/year)	(cy^3/year)	(m^3/year)	(cy^3/year)	(m^3/year)	(cy^3/year)
June 4, 2003 – May 17, 2004 = 348 days	7,356	9,563	-3,462	-4,501	3,894	5,062
July 19, 2004 – June 27, 2005 = 343 days	20,732	26,952	-610	-793	20,122	26,159
January 12, 2006 – May 26, 2006 = 134 days	24,096	31,325	-4,181	-5,435	19,915	25,890

6.4.5 Wave Directional Sensitivity

As shown in Exhibit 9, the area at the Salt Ponds inlet entrance is subjected to waves arriving both from the north and from the south. A directional sensitivity test was performed to ascertain relative exposure of the inlet to diffracted waves arriving from different directions. Sensitivity simulations were made for 3.28 ft (1 m), 5 second (s) wind-waves arriving from the northeast sector 22.5°N, east sector 90°N and southeast sector 157.5°N. Note that 5 s is the modal period of wind-waves in the region. Exhibit 14 depicts the simulations. Diffraction was well-simulated by the model. Based on these tests, the north jetty appears to be effective in sheltering the inlet from northeasterly waves. However, the existing jetty configuration as a whole does not appear to be effective against easterly and southeasterly waves.

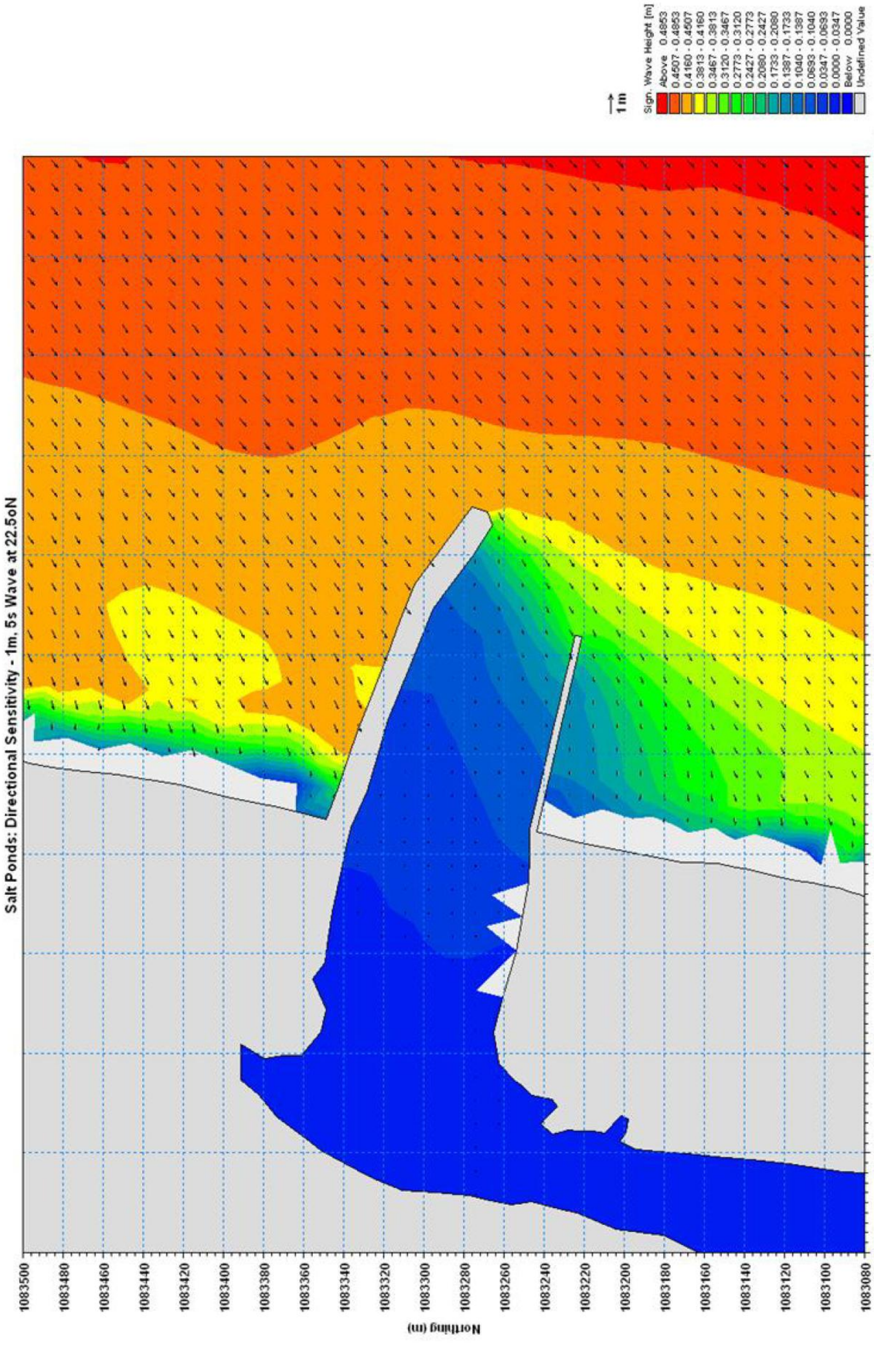


Exhibit 14a: Directional sensitivity: 3.28 ft (1 m), 5 second wind-waves approaching from 22.5ºN.

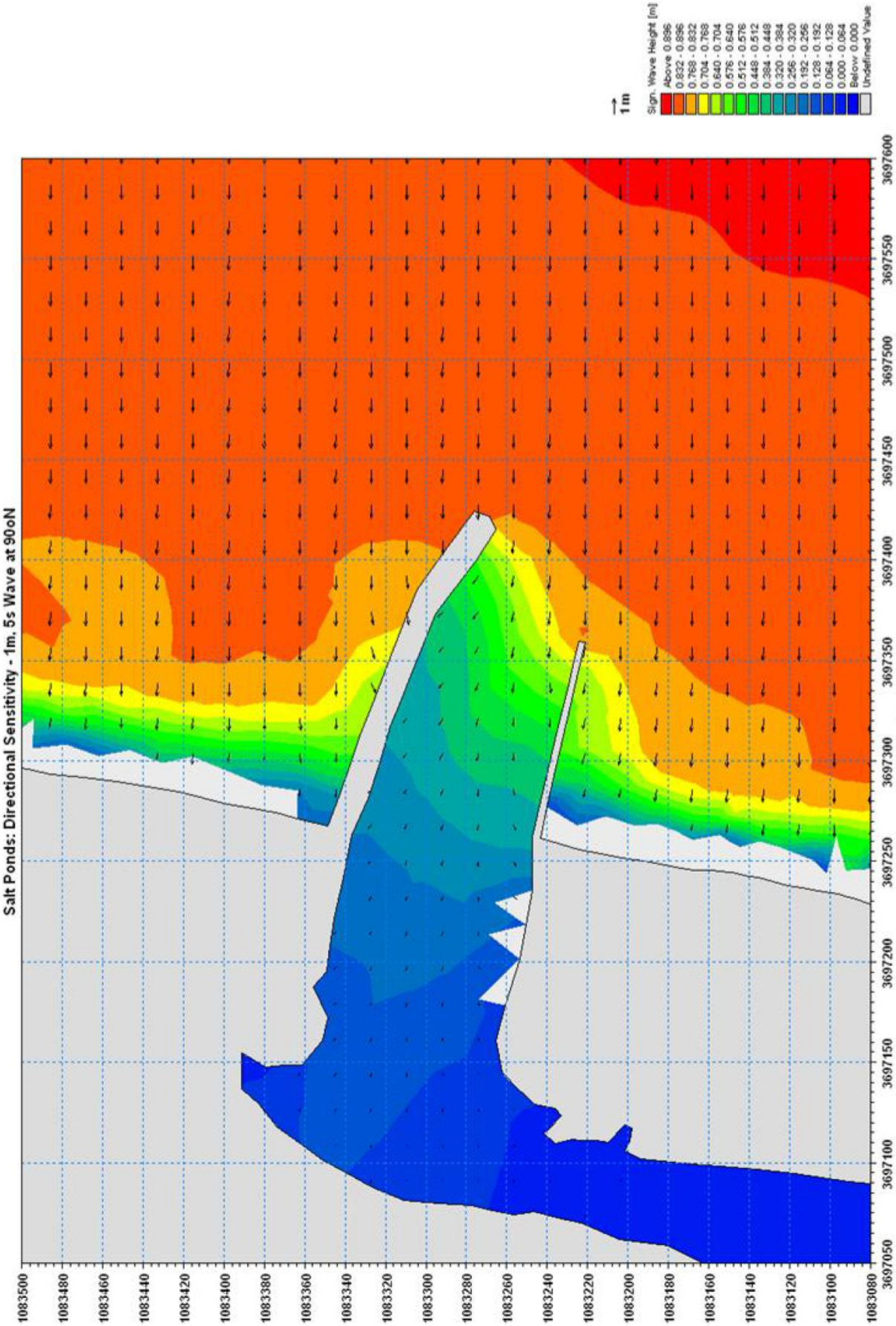


Exhibit 14b: Directional sensitivity: 3.28 ft (1 m), 5 second wind-waves approaching from 90°N.

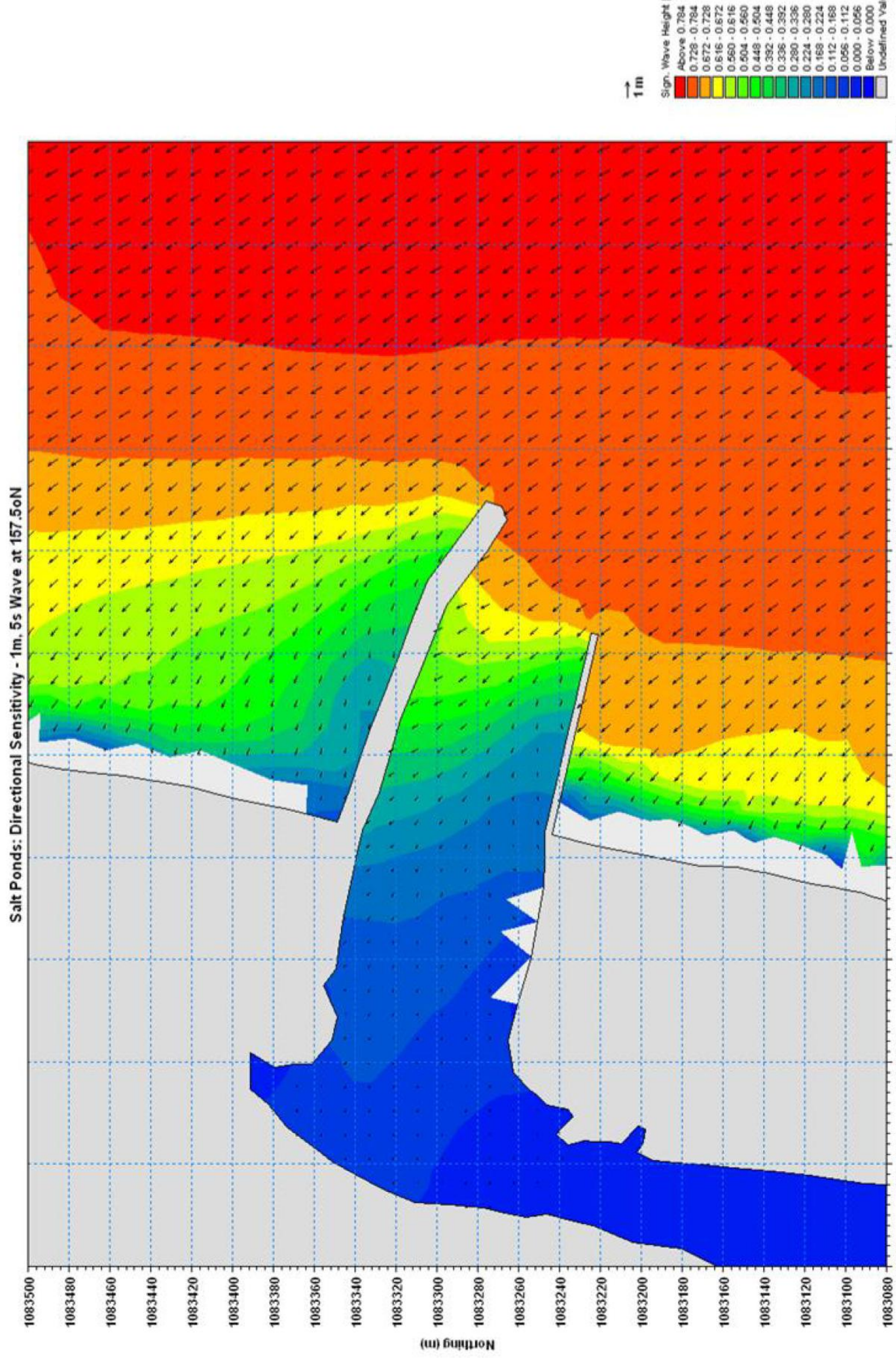


Exhibit 14c: Directional sensitivity: 3.28 ft (1 m), 5 second wind-waves approaching from 157.5°N.

6.5 Structural Alternatives

A preliminary screening of structural alternatives based on the professional experience of the consultants identified five alternative jetty and breakwater layouts for initial optimization analysis (Exhibit 15 and Table 3). Data analysis and model simulations (Exhibits 14a-c) indicated that currents are negligible inside the inlet and, in all likelihood, the inlet sedimentation—in particular near the entrance—is caused by waves incident from an envelope of directions from north to south.

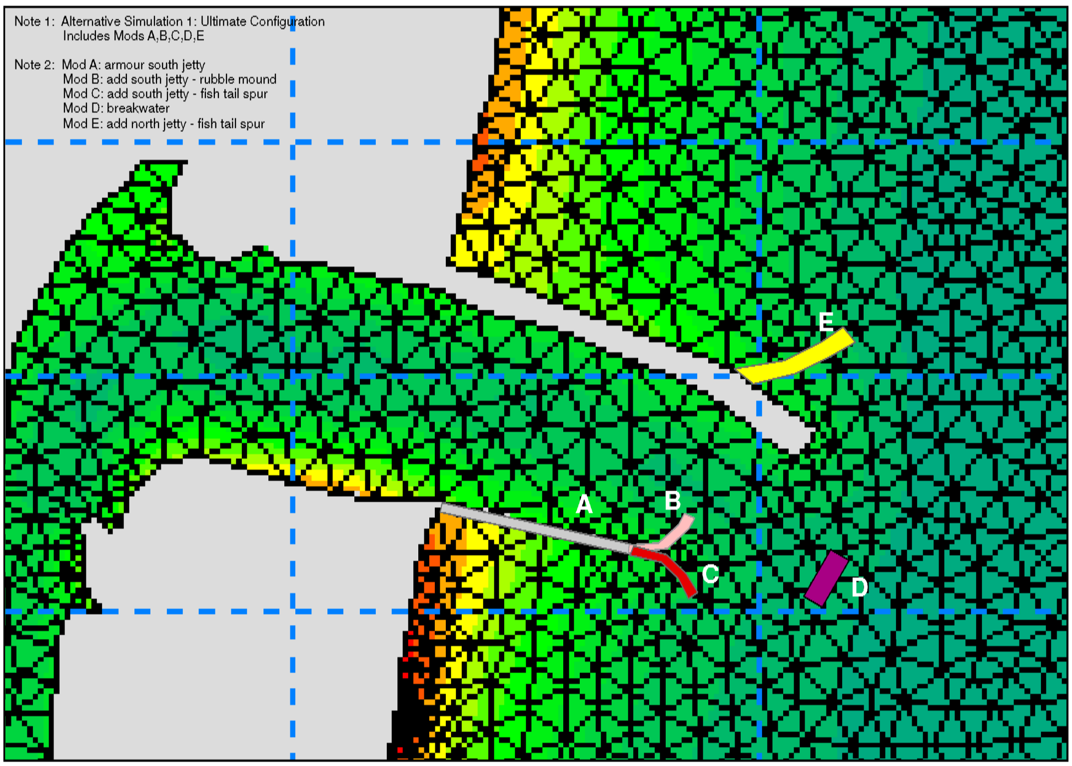


Figure 1

Exhibit 15: Initial Identification of Alternatives.

Simulations were also carried out for two additional cases in order to make comparisons and examine the relative effectiveness of the selected alternatives. In the first of the two additional simulations, the existing jetties were removed in order to determine the relative effectiveness of the current structures. The second was an 'As-Is' case with the north and south jetties in their existing configuration. This simulation was conducted to verify model performance.

Based on the simulation results for Alternatives 1 and 2, Alternatives 3 through 5 were determined to be of no additional advantage with regard to a significant reduction in shoaling. As discussed in Section 5.6, sedimentation inside the inlet is concentrated in two problem areas: near the entrance (controlled primarily by the wave and tidal power ratios) and near the narrow-neck area (see Exhibit 11 for location). Sedimentation in the narrow-neck area is controlled by the reduction in channel width, which works virtually as a partially constricted valve. Sedimentation caused by this constriction could be eliminated by widening the narrow-neck, which would require dredging impacts to the adjacent tidal marsh area on the west side of the inlet channel. Based on the type of wetlands involved (Type 1) and in the absence of a compelling demonstration of need, securing a permit for such impacts might be difficult.

Based on the performance of the five alternatives selected for this study, three additional options were also simulated: (1) widening of the channel narrow-neck (Alternatives 6 & 7), (2) construction of a suitably spaced groin-field along the north and south beaches (Alternative 8), and (3) extension of the existing jetties (Alternatives 9 & 10). Alternative 11 represents an elongation of the proposed alternative breakwater 'D' (Exhibit 15) as an alternative to a free standing off-shore breakwater. The intent is to provide more complete protection of the inlet entrance against directly incident waves.

Table 3 below shows the matrix of alternative jetty configurations.

Table 3: Matrix of Alternative Jetty and Breakwater Configurations.

Configurations	North Jetty	A	B	C	D	E	Narrow-neck widening	Groin field	Jetty Extension
Jetties Removed									
As Is	X	[1]X							
Alternative 1	X	X	X	X	X	X			
Alternative 2	X	X	X	X		X			
*Alternative 3	X	X	X		X				
*Alternative 4	X	X	X	X					
*Alternative 5	X	X		X		X			
Alternative 6	X	X					X		
Alternative 7	X	X		X		X	X		
Alternative 8	X	X						X	
Alternative 9	X	X					X		[2]X
Alternative 10	X	X					X		[3]X
Alternative 11	X	X	X	X	[4]X	X			

*These alternatives were not simulated because they were of no appreciable additional advantage.
[1]South jetty is unarmored as the present condition represents.
[2]Straight extension of both the jetties.
[3]Bent extension of the north jetty.
[4]D is elongated.

6.6 Simulations

A total of 13 scenarios were simulated for both confidence building exercises in model performance and for optimization of alternatives. Scenarios 1 and 2 were base conditions against which other simulations are compared. Scenarios 2, 3, 4 and 6 applied the September 2003 Hurricane Isabel and the April 1991 northeaster (See section 4.2.2, Exhibit 7b) and were used for examining model performance. Table 4 below shows the simulation scenarios evaluated.

Table 4: Description of Simulation Scenarios

Scenario	Boundary Forcing (-)	Model (-)	Model description
1	3.5-day Feb93 local storm	As Is	Existing jetties model
2	4-day Sep03 Hurricane Isabel	As Is	Existing jetties model
3	4-day Sep03 Hurricane Isabel	No Jetties	Removed existing jetties model
4	4-day Sep03 Hurricane Isabel	Alternative 1	Alternative 1 model (Table 3)
5	3.5-day Feb93 local storm	Alternative 1	Alternative 1 model (Table 3)
6	3.5-day Apr91 local storm	Alternative 1	Alternative 1 model (Table 3)
7	3.5-day Feb93 local storm	Alternative 2	Alternative 2 model (Table 3)
8	3.5-day Feb93 local storm	Alternative 6	Alternative 6 model (Table 3)
9	3.5-day Feb93 local storm	Alternative 7	Alternative 7 model (Table 3)
10	3.5-day Feb93 local storm	Alternative 8	Alternative 8 model (Table 3)
11	3.5-day Feb93 local storm	Alternative 9	Alternative 9 model (Table 3)
12	3.5-day Feb93 local storm	Alternative 10	Alternative 10 model (Table 3)
13	3.5-day Feb93 local storm	Alternative 11	Alternative 11 model (Table 3)

6.6.1 Simulation – “As Is” Existing Jetty Configuration

Scenarios 1 and 2 represented “As Is” models and were forced by the February 1993 local storm and the September 2003 Hurricane Isabel, respectively.

Simulation snapshots of wave height, current and sand transport are shown in Exhibits 16a, b and c and Exhibit 17. The snapshot depicted in Exhibit 16a represents falling tide on February 1993 at 12:15, immediately after high water. At that time, tide level was 4 ft (1.22 m) above MLLW, H_s = 4.3 ft (1.3 m), T_p = 5.6 s and MWD = 70°N (Exhibit 16). This snapshot of east-northeasterly waves indicates the nature of wave transformation (refraction, shoaling, diffraction and concentration of wave energies at the jetties), wave and tide induced currents and longshore transports and their relative distribution.

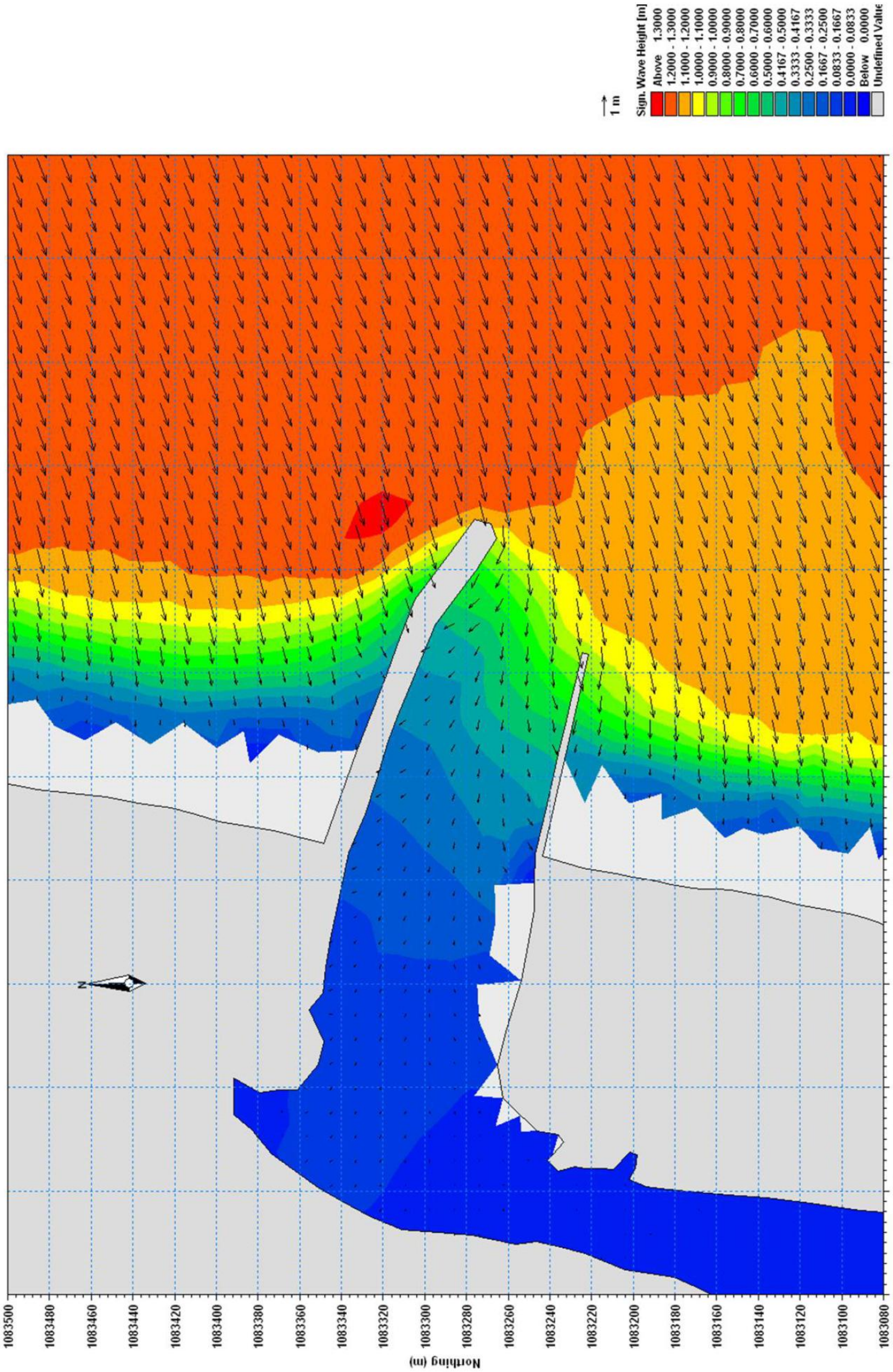


Exhibit 16a: Significant wave height snapshot for scenario 1 representing falling tide on February 27, 1993 at 12:15:00.

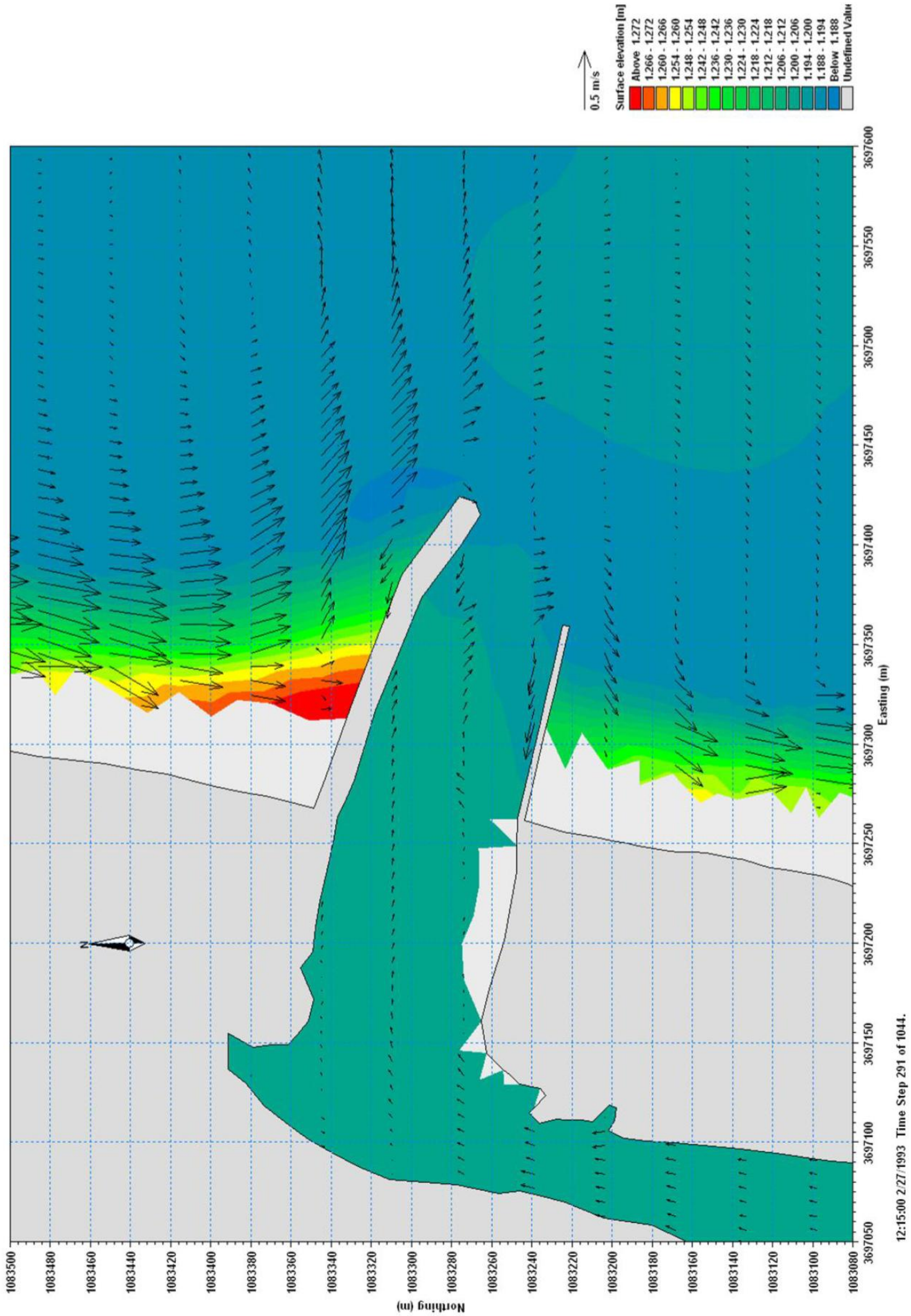


Exhibit 16b: Depth-average current snapshot for scenario 1 representing falling tide on February 27, 1993 at 12:15:00.

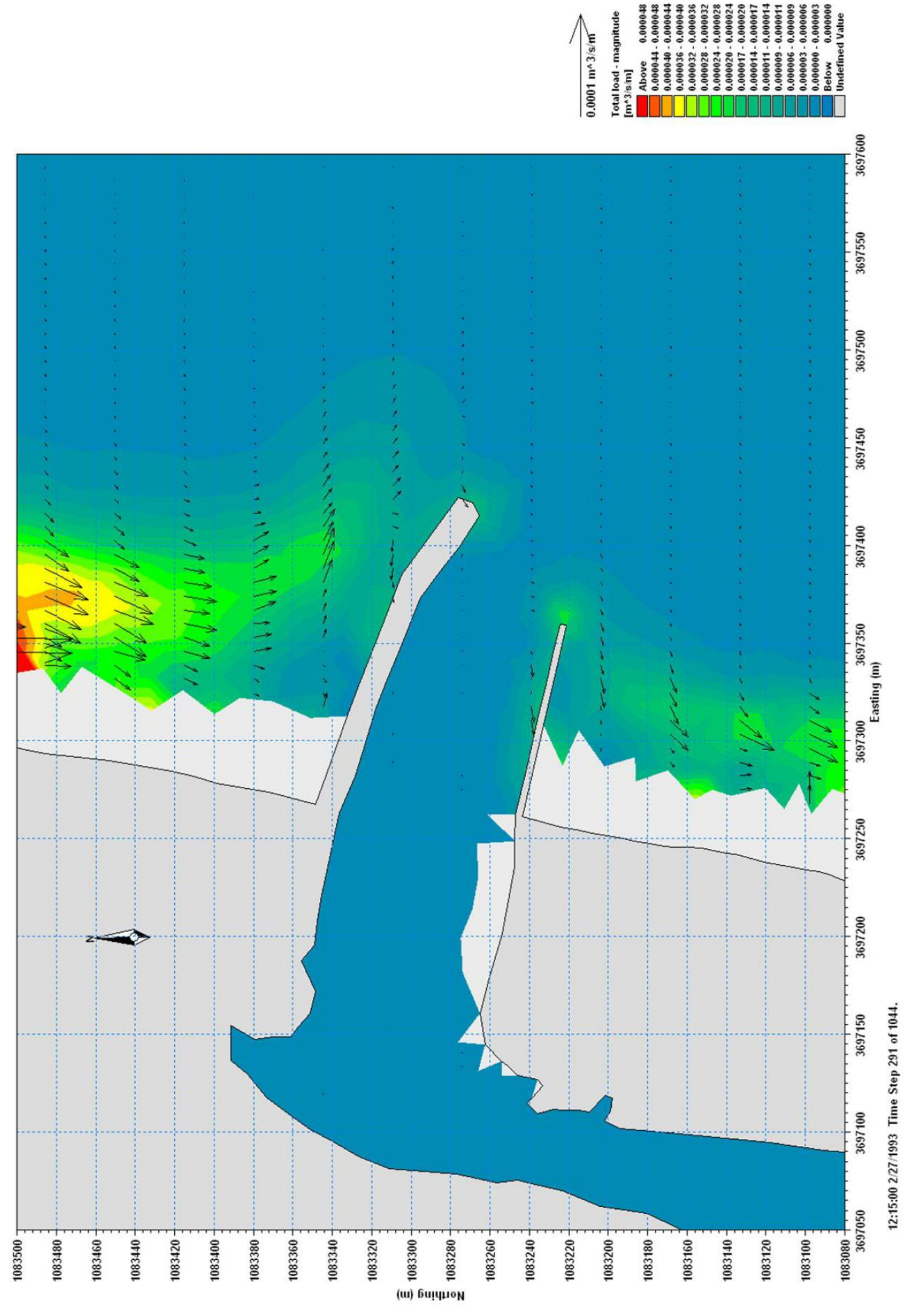


Exhibit 16c: Total load transport snapshot for scenario 1 representing falling tide on February 27, 1993 at 12:15:00.

Exhibits 17a-c show another snapshot during the same storm period with swells coming from the southeast. The snapshot represents falling tide on March 1, 1993 at 15:00, right after high water. During that time, tide level was 3 ft (0.9 m) above MLLW, H_s = 2.3 ft (0.72 m), T_p = 15.17 s and MWD = 133°N (Exhibit 16). And examination of Exhibits 15 and 16 shows that the present jetty configuration is more vulnerable to southeasterly swells than to northeasterly wind-waves. The formation of eddies both in and outside of the inlet entrance is notable, especially during southeasterly swells. These eddies play an important role in erosion/scour and sedimentation.

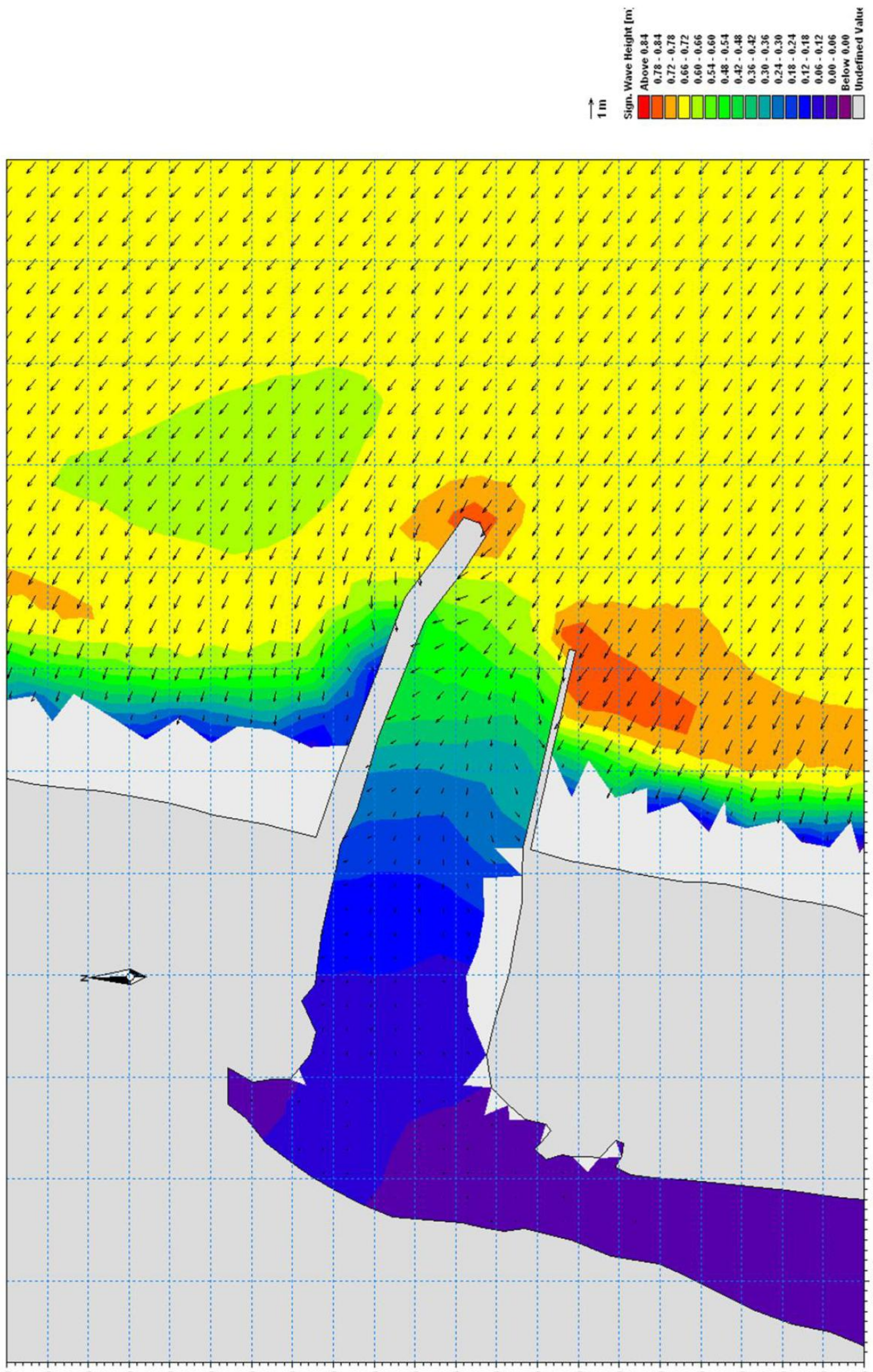


Exhibit 17a: Significant wave height snapshot for scenario 1 representing falling tide on March 1, 1993 at 15:00:00.

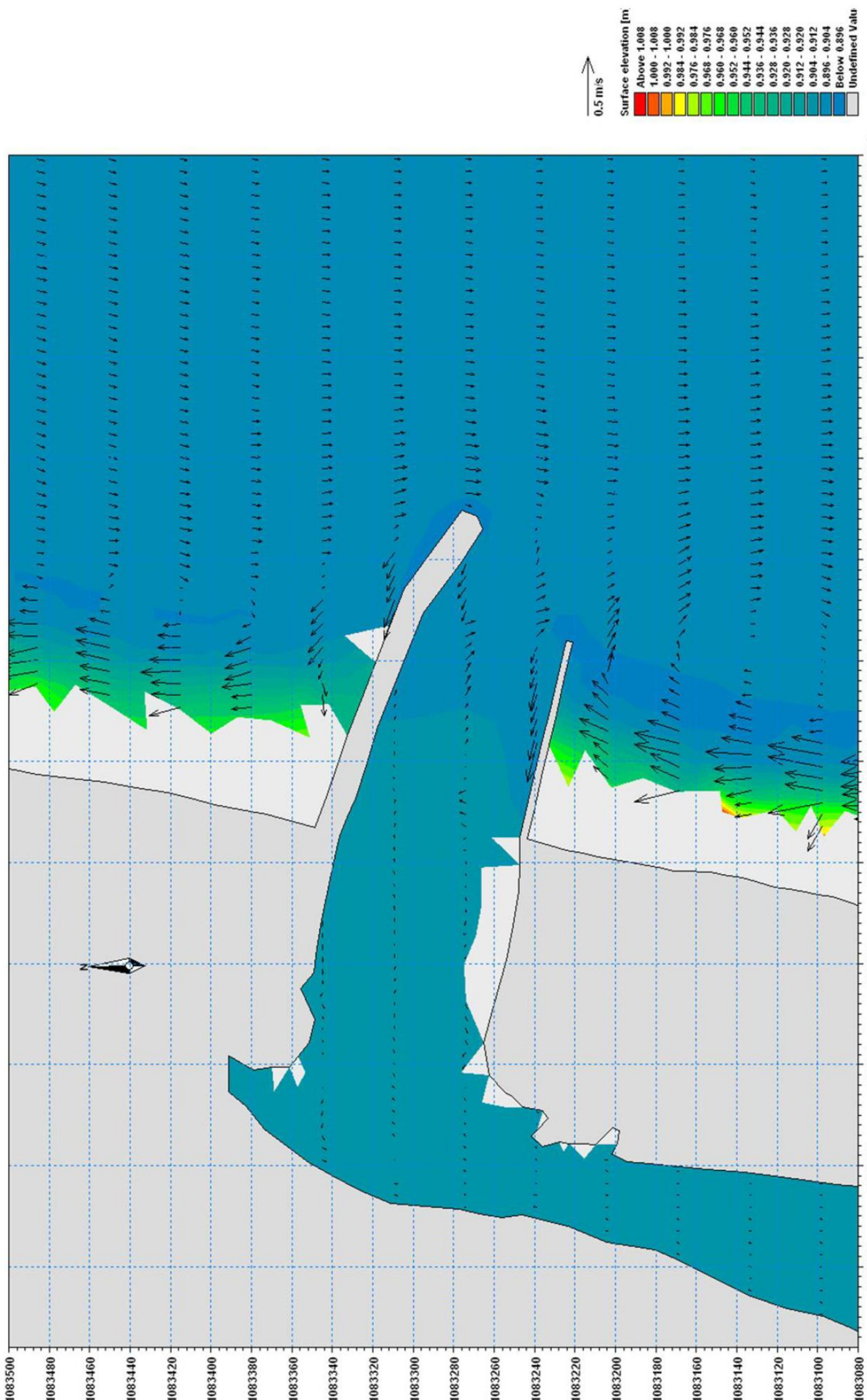


Exhibit 17b: Depth-average current snapshot for scenario 1 representing falling tide on March 1, 1993 at 15:00:00.

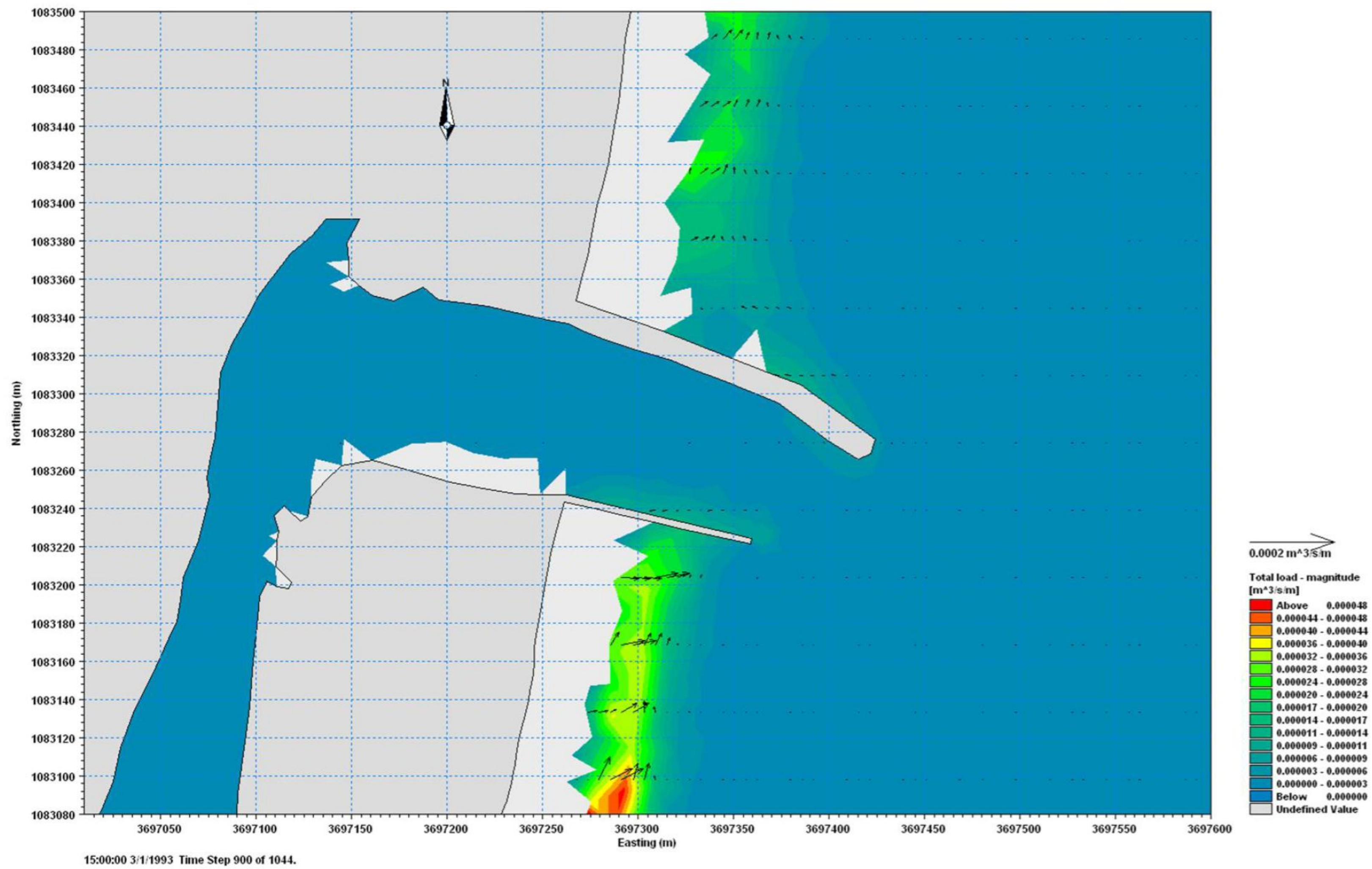


Exhibit 17c: Total load transport snapshot for scenario 1 representing falling tide on March 1, 1993 at 15:00:00.

The bed level (bottom bathymetry) changes caused by the February 1993 storm and the September 2003 Hurricane Isabel (Scenarios 1 and 2, respectively) are shown in Exhibits 18a and b. Note that erosion and sedimentation patterns are different for the two events. During Hurricane Isabel a spit is formed at the end of the north jetty together with an ebb-tidal delta.

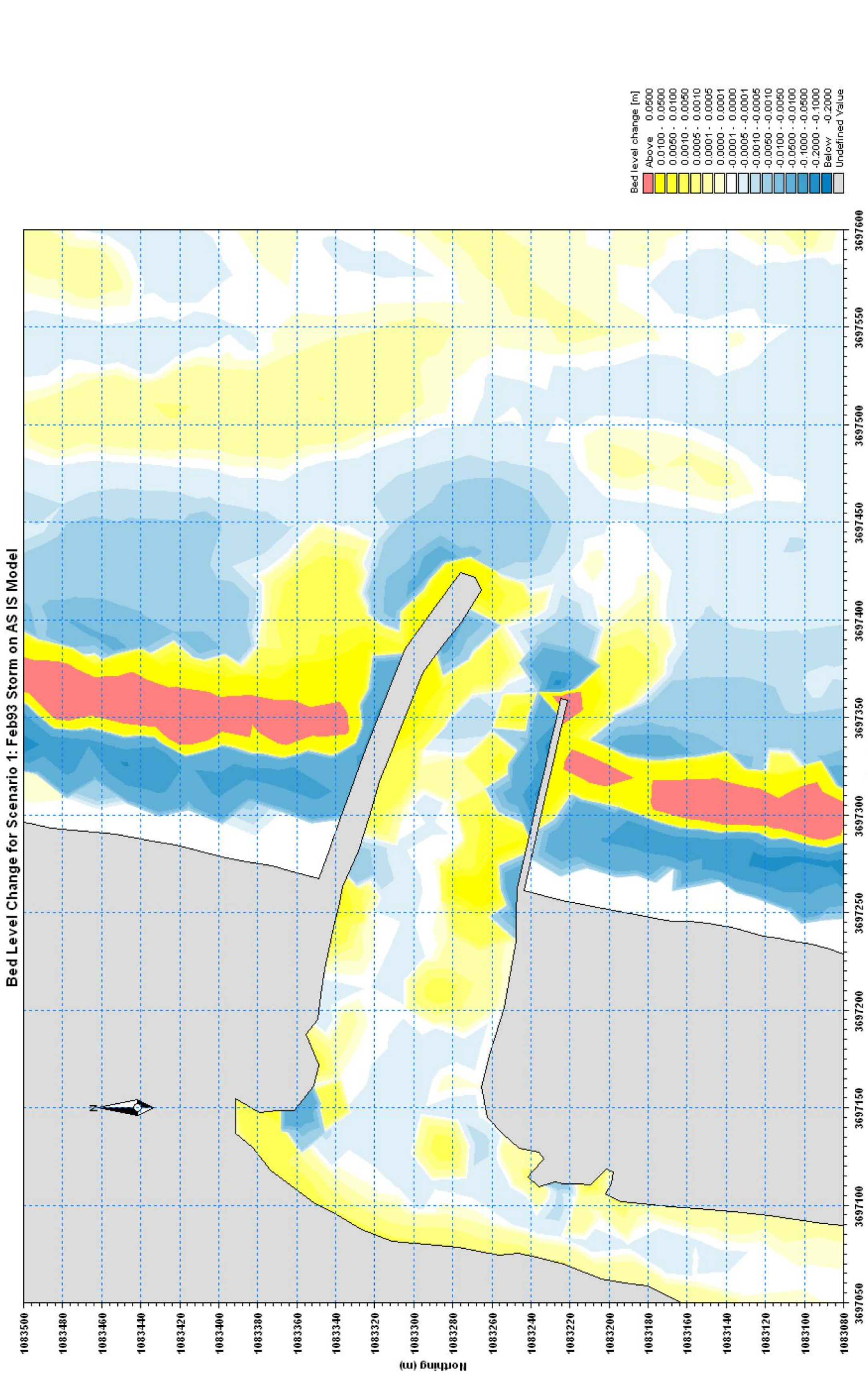


Exhibit 18a: Bed level changes for As Is Model scenario 1 simulation.

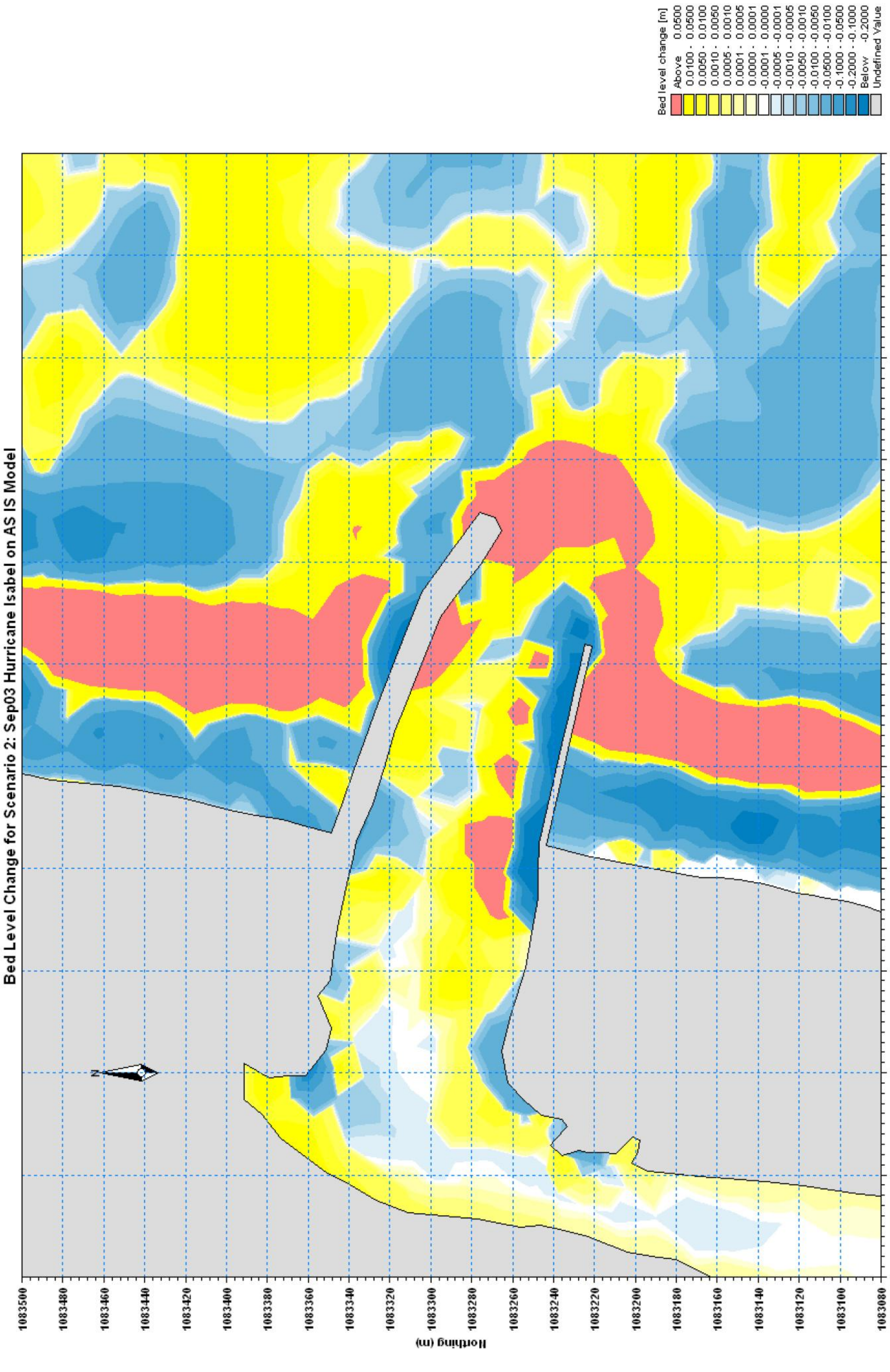


Exhibit 18b: Bed level changes for As Is Model scenario 2 simulation.

Two approaches were applied to evaluate the effectiveness of the alternatives in minimizing sedimentation inside the inlet. The first was a comparison of changes in net shoaling volumes following the simulation of a common event. The inlet area bounded by sections a-a' and h-h' (Exhibit 19) was used for this purpose. An evaluation based solely on net volume change can be misleading because areas of navigation-hindering sedimentation in the channel may not be completely reflected by such an approach. The second approach to alternative evaluation involved superimposed cross-sections taken at sections m-m', a-a', a-b, k-k', c-c' and d-d' (Exhibit 19). As discussed later, this approach appears to be more appropriate considering the project goal of maintaining navigability within the inlet channel while reducing maintenance dredging frequency.

Sections J-J', K-K', L-L' and M-M' depict longshore transport along the shorelines adjacent to the inlet entrance.

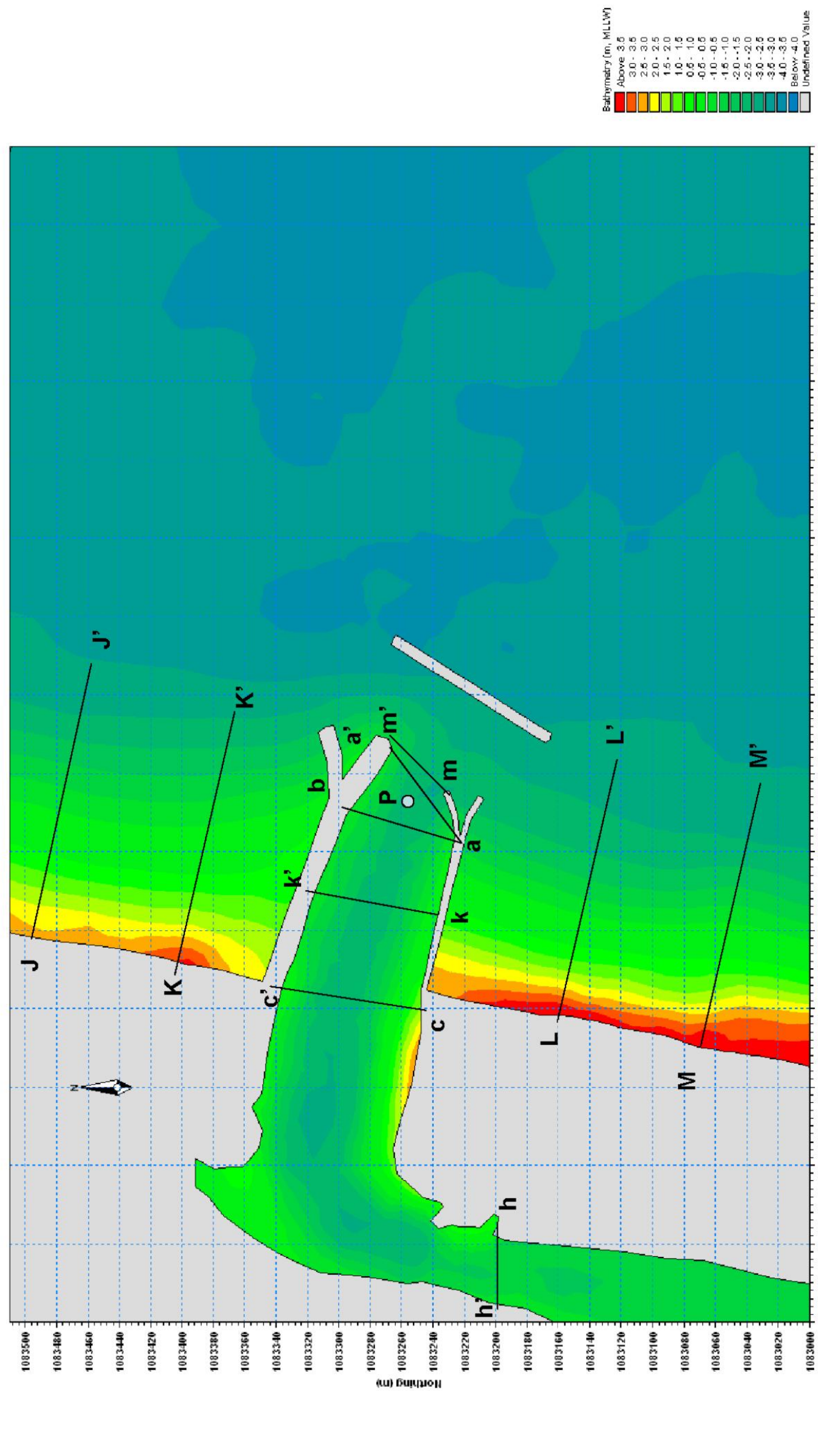


Exhibit 19: Index of locations of points and sections. Volume estimates represent inlet area bounded by sections aa' and hh'.

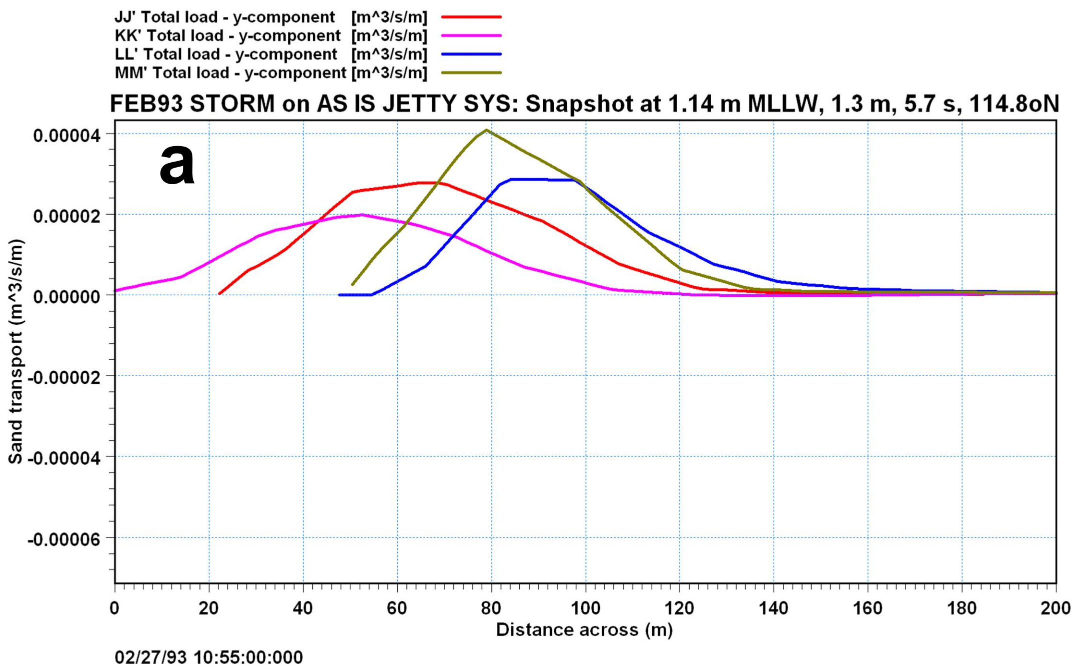
Table 5 provides a summary of shoaling, erosion and net volume changes for the 13 simulated scenarios included in this study. A comparison of the volume changes (Scenarios 1 and 2) caused by the two storms indicates that during Hurricane Isabel the volumetric change is an order of magnitude higher than during the February 1993 storm. The Hurricane Isabel simulations produced net erosion within the comparison area due primarily to erosion occurring along the inlet side of the south jetty (Exhibit 18b). This appears to be the reason for the lower shoaling rate observed during Hurricane Isabel (Table 5).

Exhibit 20 shows simulated longshore transport (y-component) snapshots for the different transects depicted in Exhibit 19. The presence of transport gradients between transects results in erosion or sedimentation. For example, southeasterly waves cause sedimentation at the south beach and erosion at the north beach (Exhibit 20a). Similarly, northeasterly waves cause sedimentation at the north beach and erosion at the south beach (Exhibit 20b).

Table 5: Model Simulated Volume Changes

Scenario	Volume Change					
	Erosion		Shoaling		Net	
	(m^3)	(yd^3)	(m^3)	(yd^3)	(m^3)	(yd^3)
1	-94	-123	334	437	*240	314
2	-2944	-3851	843	1103	-2101	-2748
3	-4255	-5565	3600	4709	-655	-857
4	-1133	-1482	222	290	-911	-1192
5	-61	-80	39	51	-22	-29
6	-264	-345	47	61	-217	-284
7	-51	-67	29	38	-22	-29
8	-81	-106	330	432	249	326
9	-215	-281	62	81	-153	-200
10	-40	-52	228	298	188	246
11	-33	-43	49	64	16	21
12	-63	-82	105	137	42	55
13	-8	-10	14	18	6	8
*The net shoaling rate for the 3.5-day storm period when translated to the annual volumetric change amounts to 25,029 m^3/year (32,373 yd^3/year).						

Inlet sedimentation is anticipated to be the result of a combination of factors: longshore transport, sand transport perpendicular to the shoreline (cross-shore transport), wind-blown sand (aeolian transport) and transport by stormwater runoff. The last two factors are presumably minor and were not accounted for in the model. During the February 1993 storm, the average northward transport was 6.5 yd^3 (5 m^3) while the average southward transport was 41.8 yd^3 (32 m^3), giving a gross transport of 48.4 yd^3 (37 m^3) and a net southward transport of 35.3 yd^3 (27 m^3). An extrapolation of the gross transport of 48.4 yd^3 (37 m^3) would represent an annual transport of about 5,050 yd^3/year (3,860 m^3/year). Since this magnitude represents approximately 20% of the inlet shoaling rate, cross-shore transport is thought to be the dominant mechanism of shoaling within the Salt Ponds inlet.



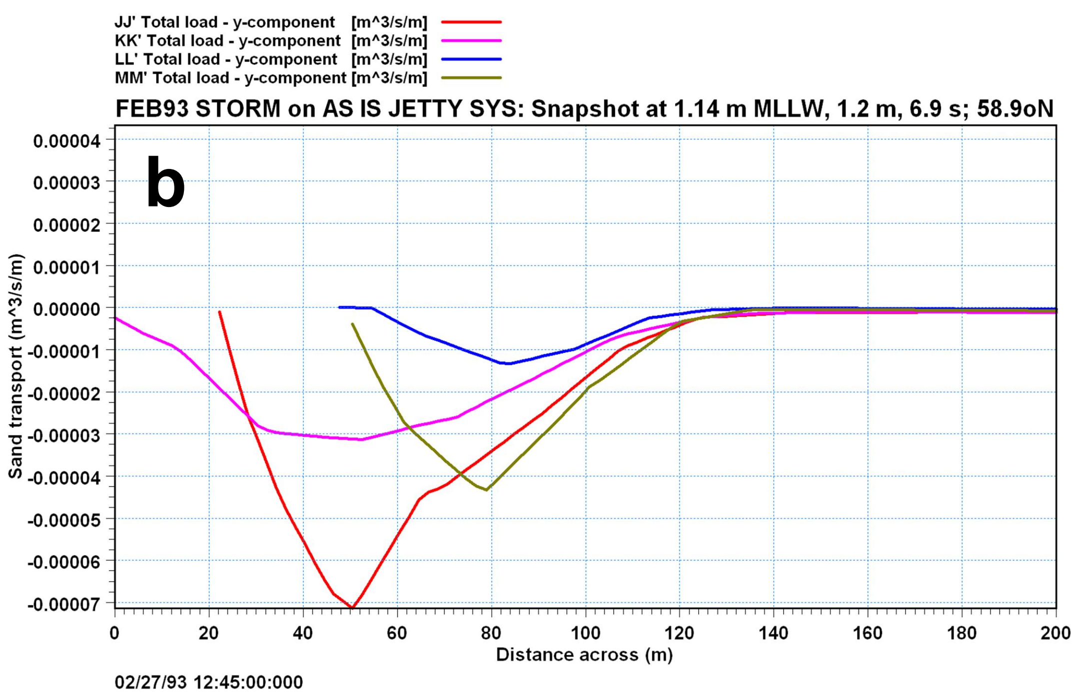


Exhibit 20: Snapshots of longshore transports during the February 1993 storm. a. Southeasterly wave incident at 115°N. b. Northeasterly wave incident at 59°N.

6.6.2 Simulation – No Jetties

In order to investigate the functionality and efficacy of the existing jetties, Scenario 3 (removal of the existing jetties) was performed. Exhibit 21 shows the simulated bed level change that would have occurred during Hurricane Isabel with an unprotected inlet. This simulation demonstrated that a single event like Hurricane Isabel could completely choke the inlet. The existing jetty configuration has prevented such an episode (Exhibit 18b). However, the present configuration still remains vulnerable to high sedimentation at the entrance during a hurricane event like Isabel.

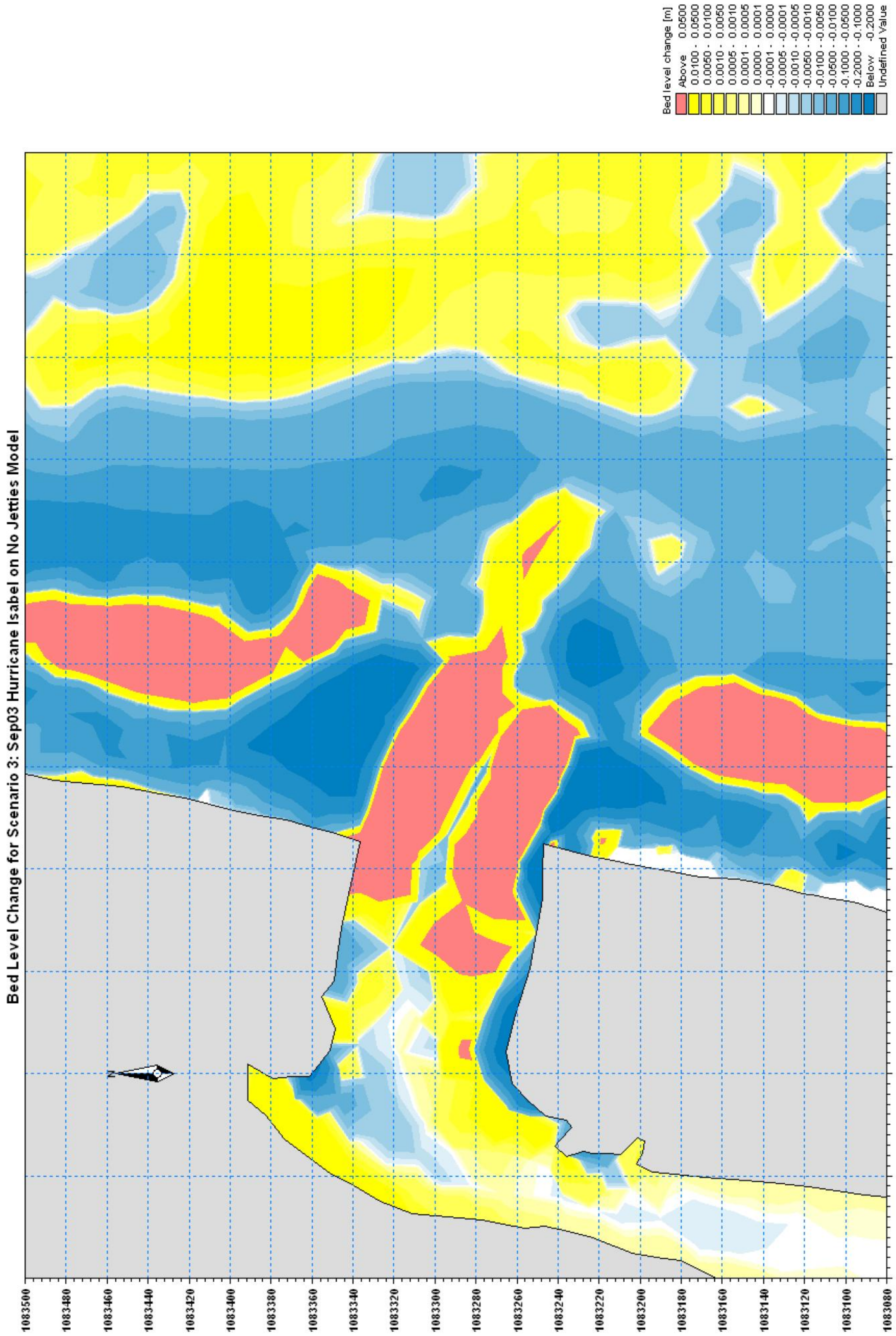


Exhibit 21: Bed-level changes for No Jetties model Scenario 3.

6.6.3 Simulation of Structural Alternatives

As discussed previously, a total of 5 alternatives were initially identified for inclusion in this study. Additional options including narrow-neck widening, groin field jetty extensions and elongation of the entrance breakwater were subsequently added to the suite of alternatives as a result of the poor performance of the initial 5 alternatives (Tables 3 and 4). The results of model simulations corresponding to the February 1993 storm for scenarios 5, 7, 8, 9, 10, 11, 12 and 13 are discussed in this section. Exhibit 22 shows the bed level changes for these scenarios. Color bands on the associated bed-level change Exhibits provide a means of comparing the result of each scenario. In these Exhibits, yellows and oranges represent sedimentation, blues depict erosion, and white indicates no change in bed level. These Exhibits are best interpreted as qualitative indications of erosion and sedimentation showing relative trends in bed level change over the model domain. Effects related to the presence of the narrow-neck are visible in the sedimentation patterns shown in Exhibits 22a and 22b. This could be alleviated by widening the narrow-neck (Exhibits 22c, 22d, 22f, and 22g); however, as noted above, this would require tidal marsh wetland impacts.

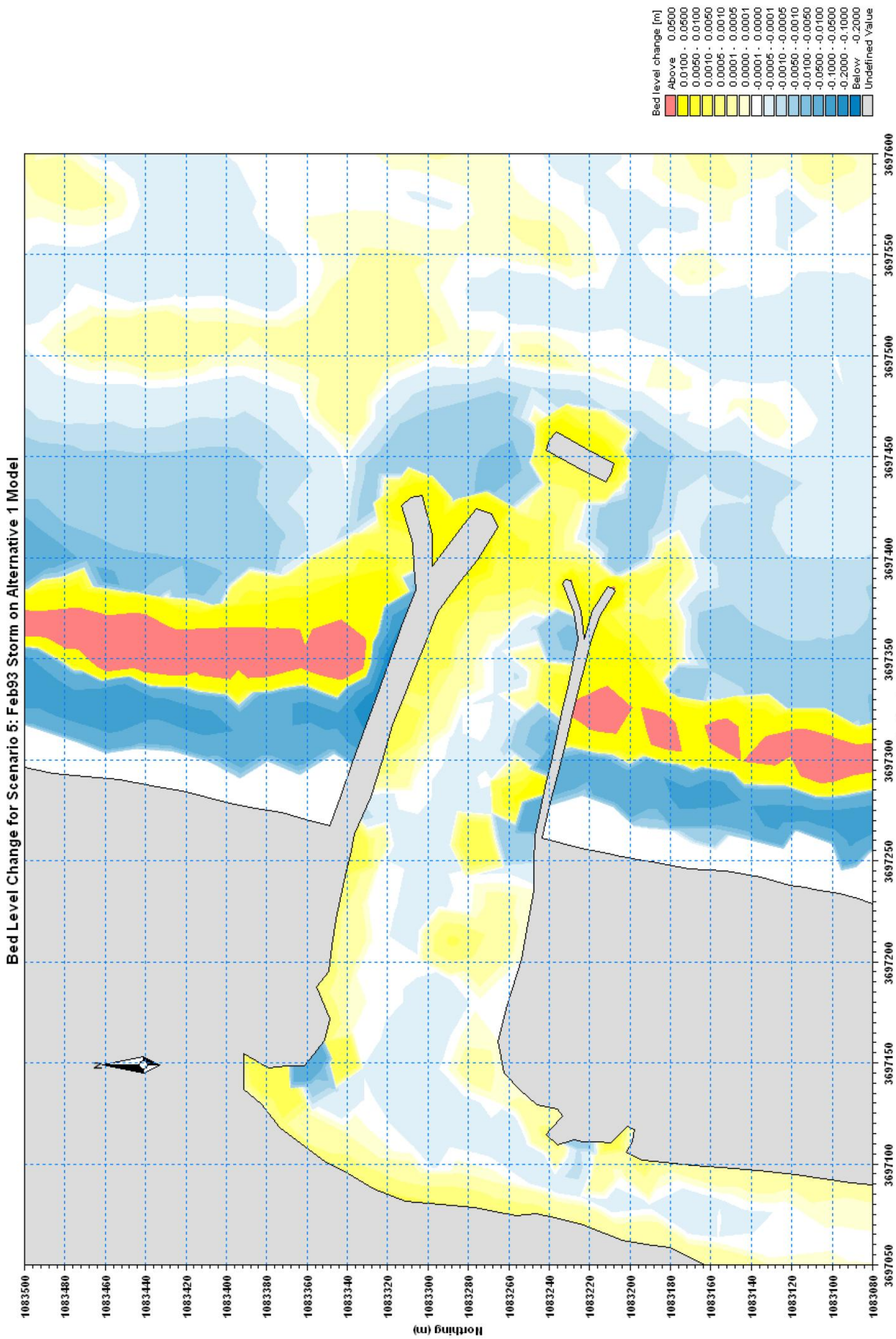


Exhibit 22a: Bed level changes for Alternative 1 Model Scenario 5.

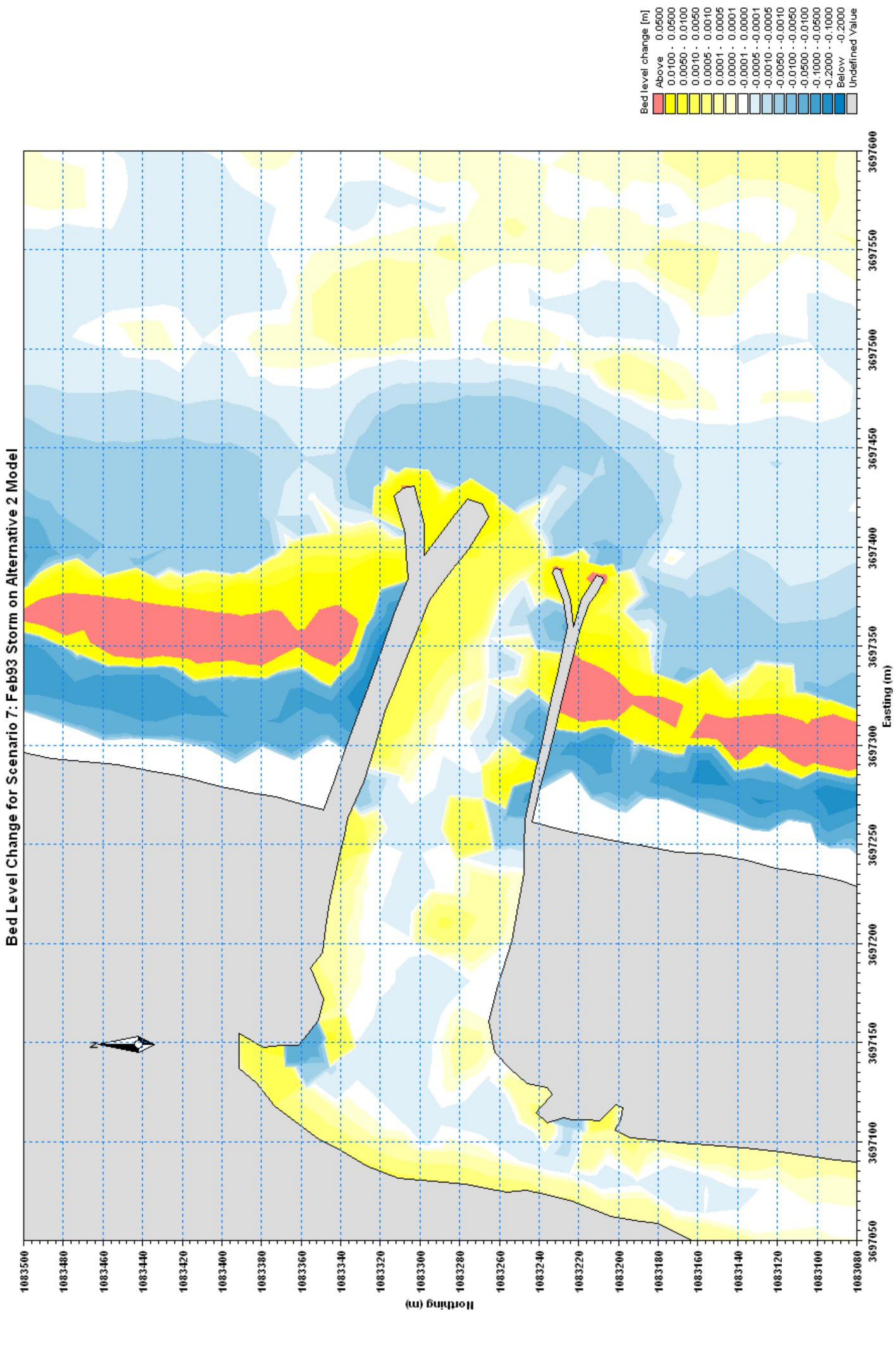


Exhibit 22b: Bed level changes for Alternative 2 Model Scenario 7.

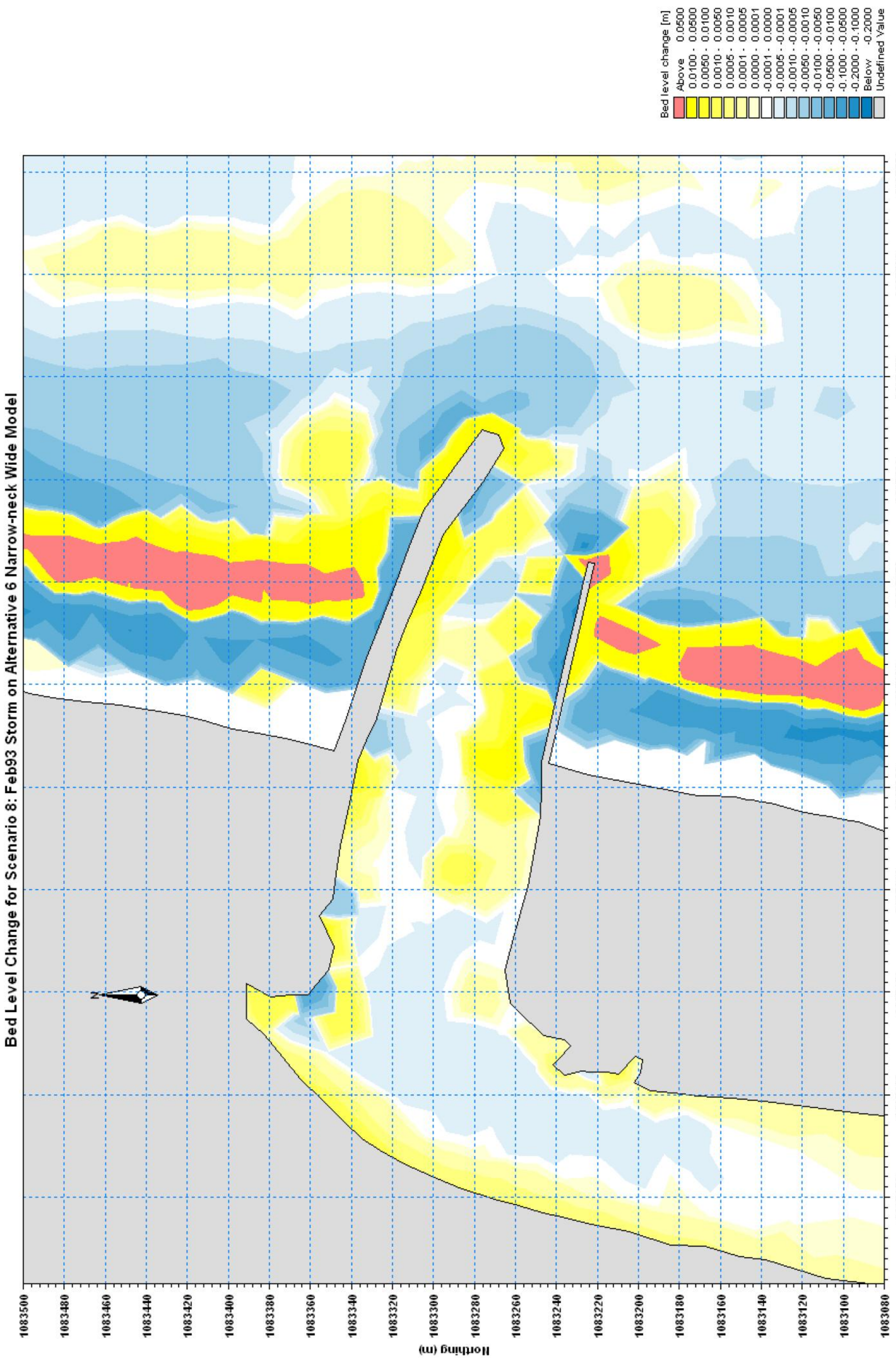


Exhibit 22c: Bed level changes for Alternative 6 Model Scenario 8.

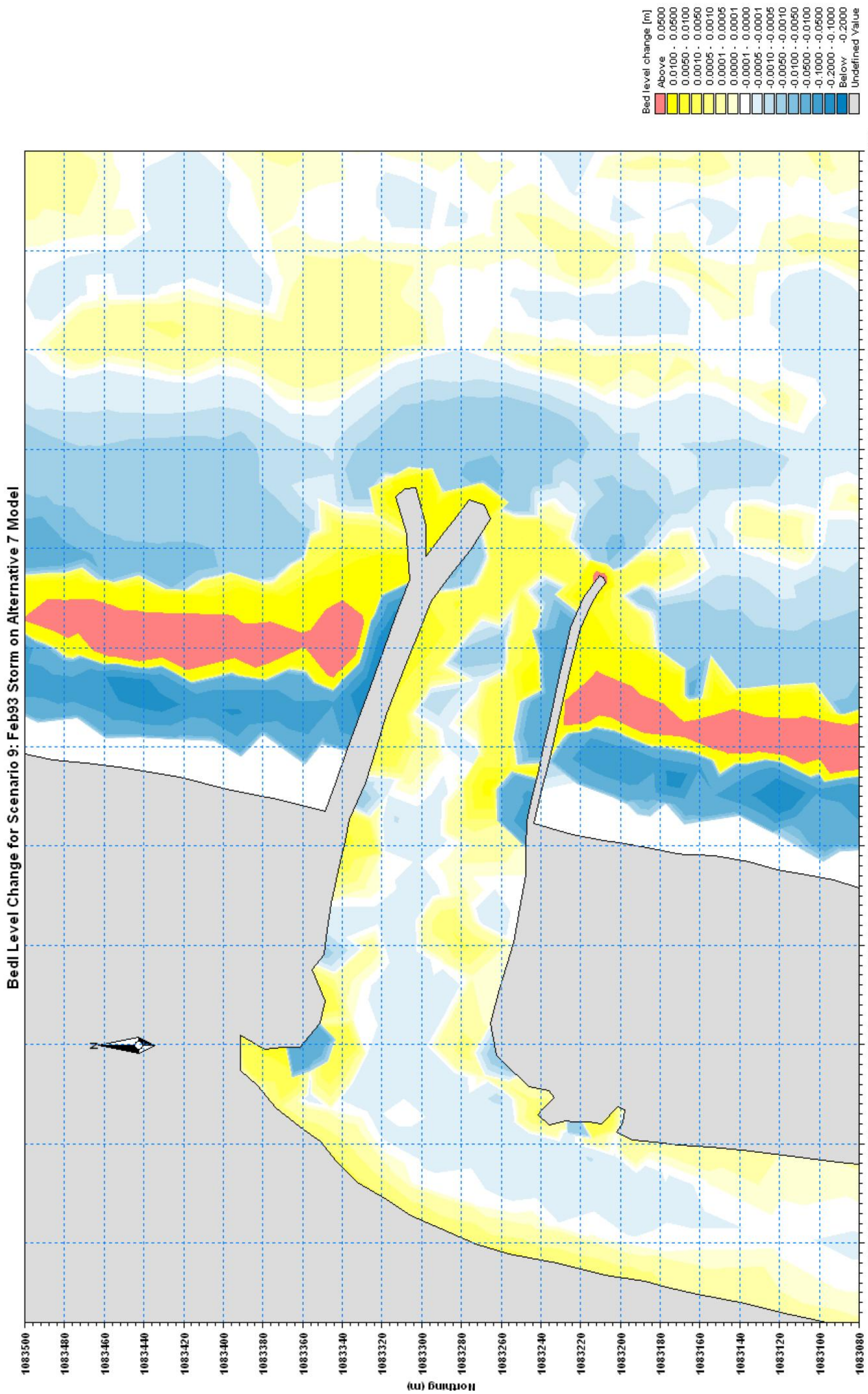


Exhibit 22d: Bed level changes for Alternative 7 Model Scenario 9.

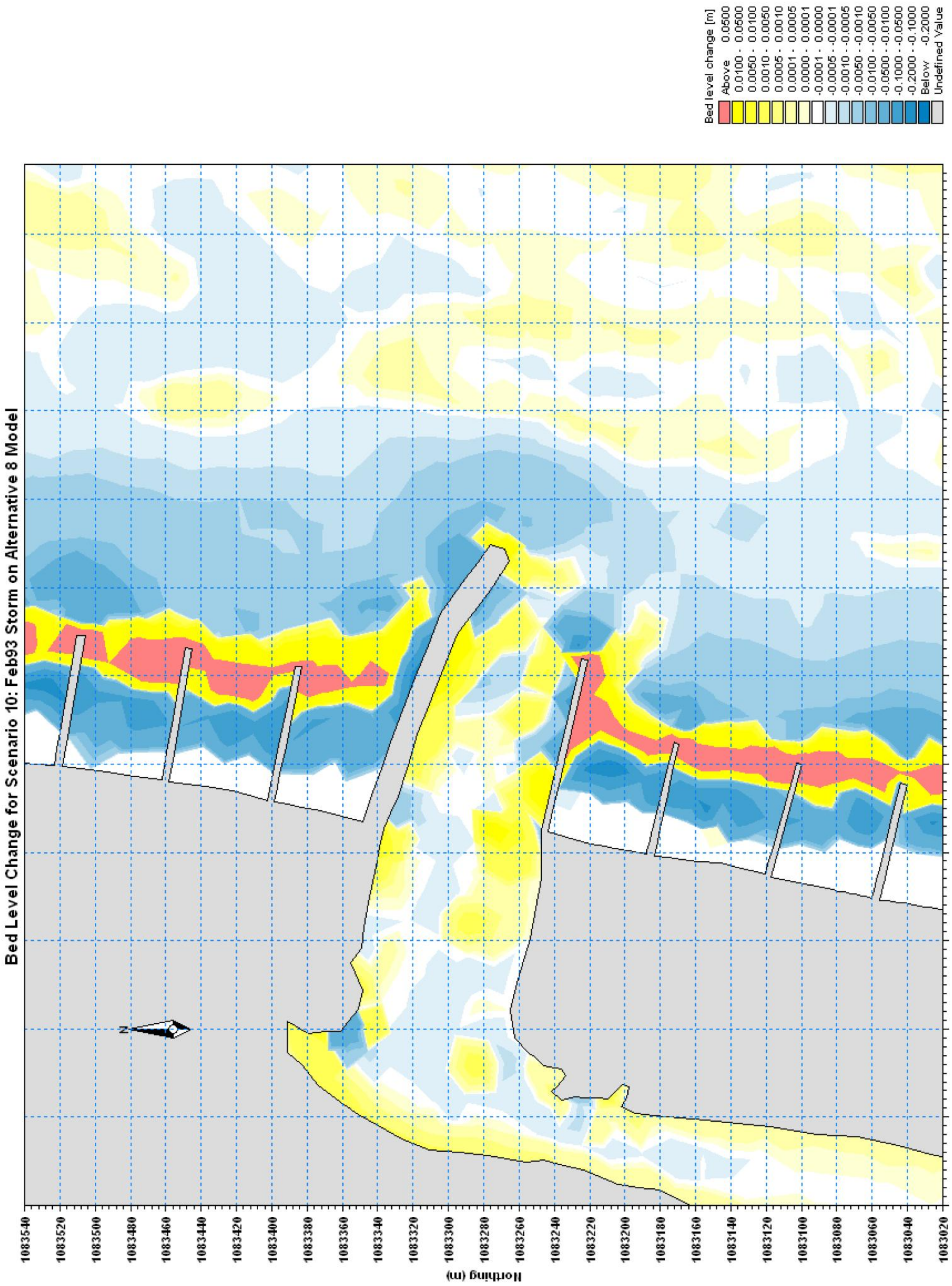


Exhibit 22e: Bed level changes for Alternative 8 Model Scenario 10.

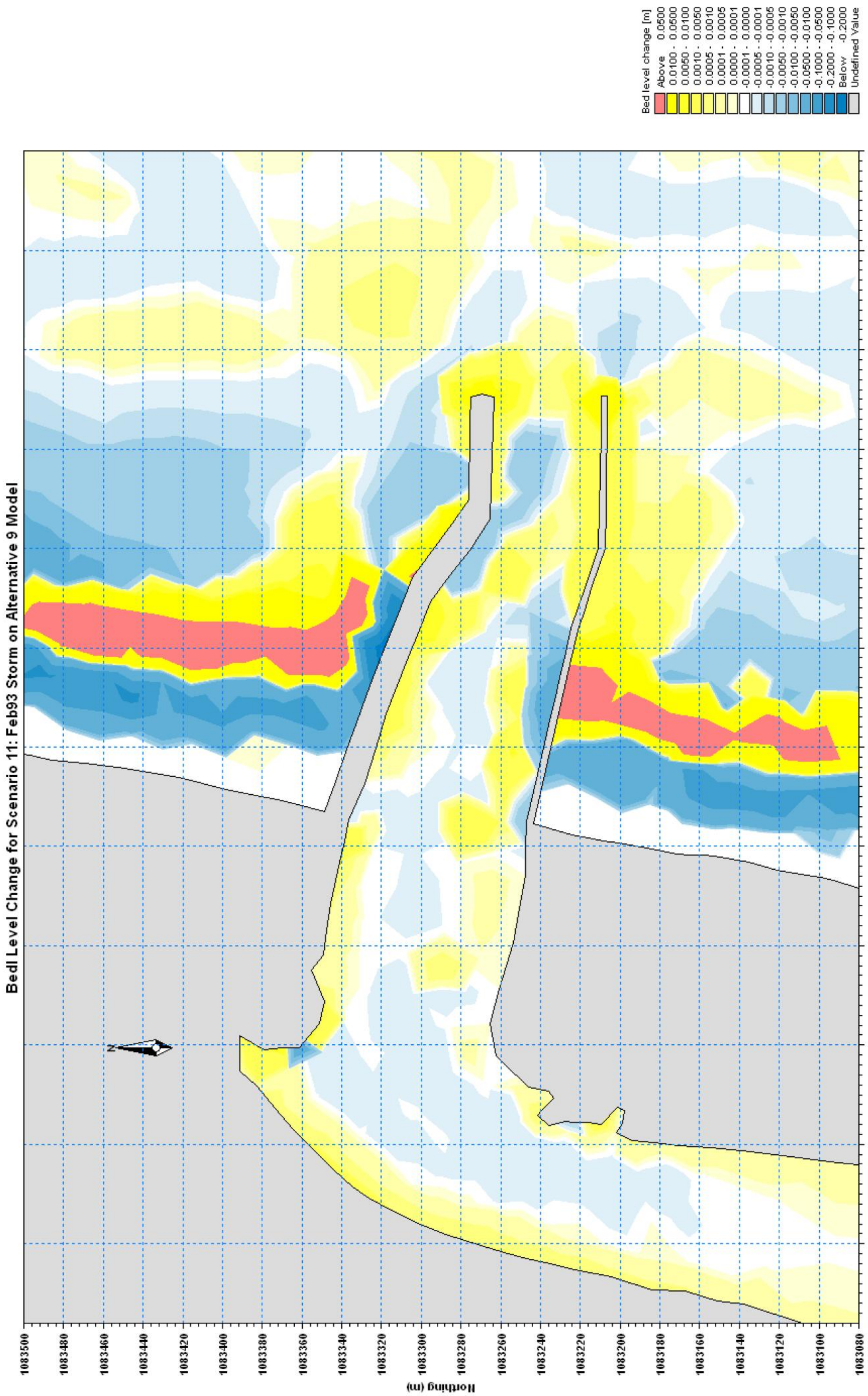


Exhibit 22f: Bed level changes for Alternative 9 Model Scenario 11.

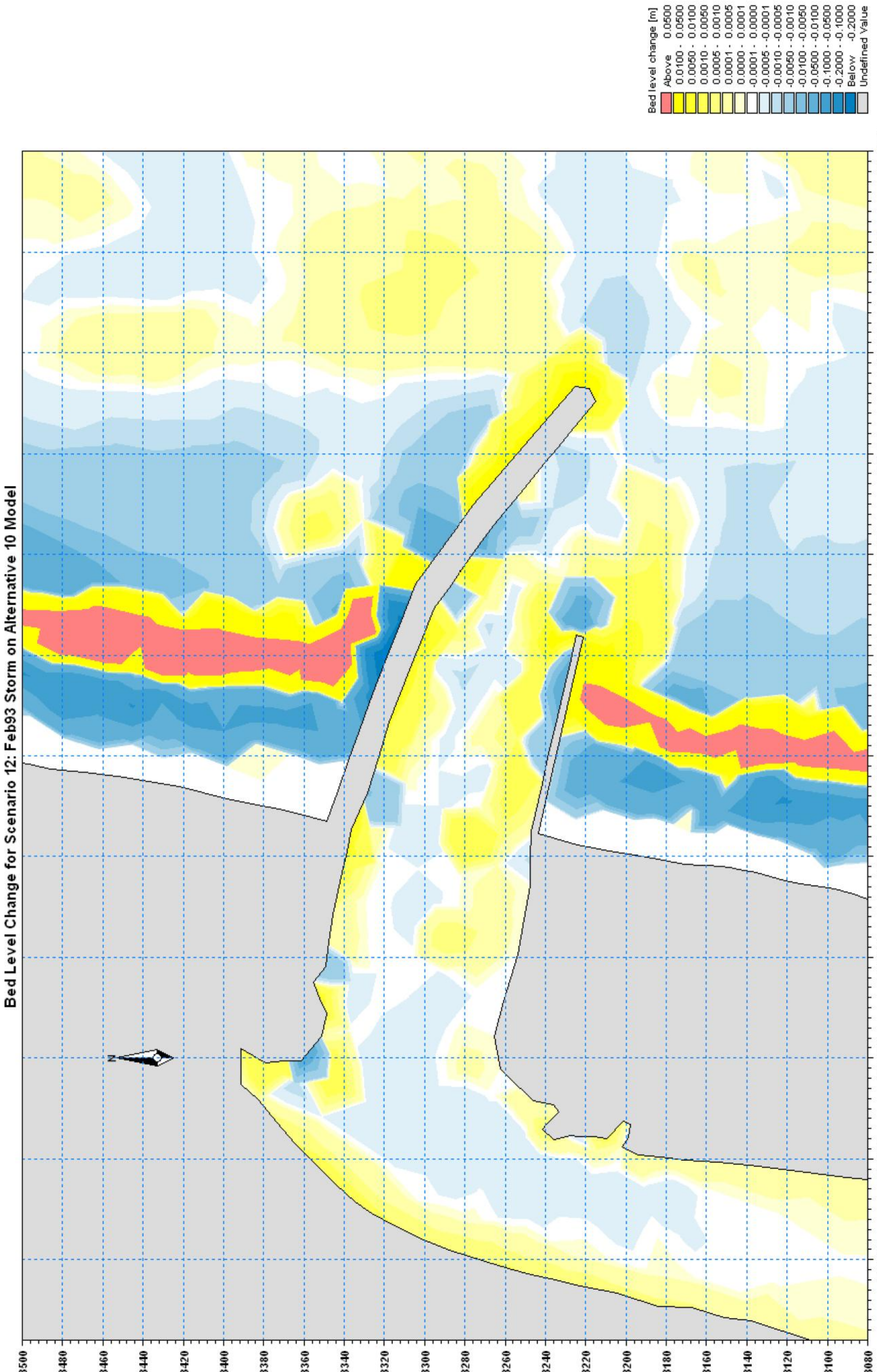


Exhibit 22g: Bed level changes for Alternative 10 Model Scenario 12.

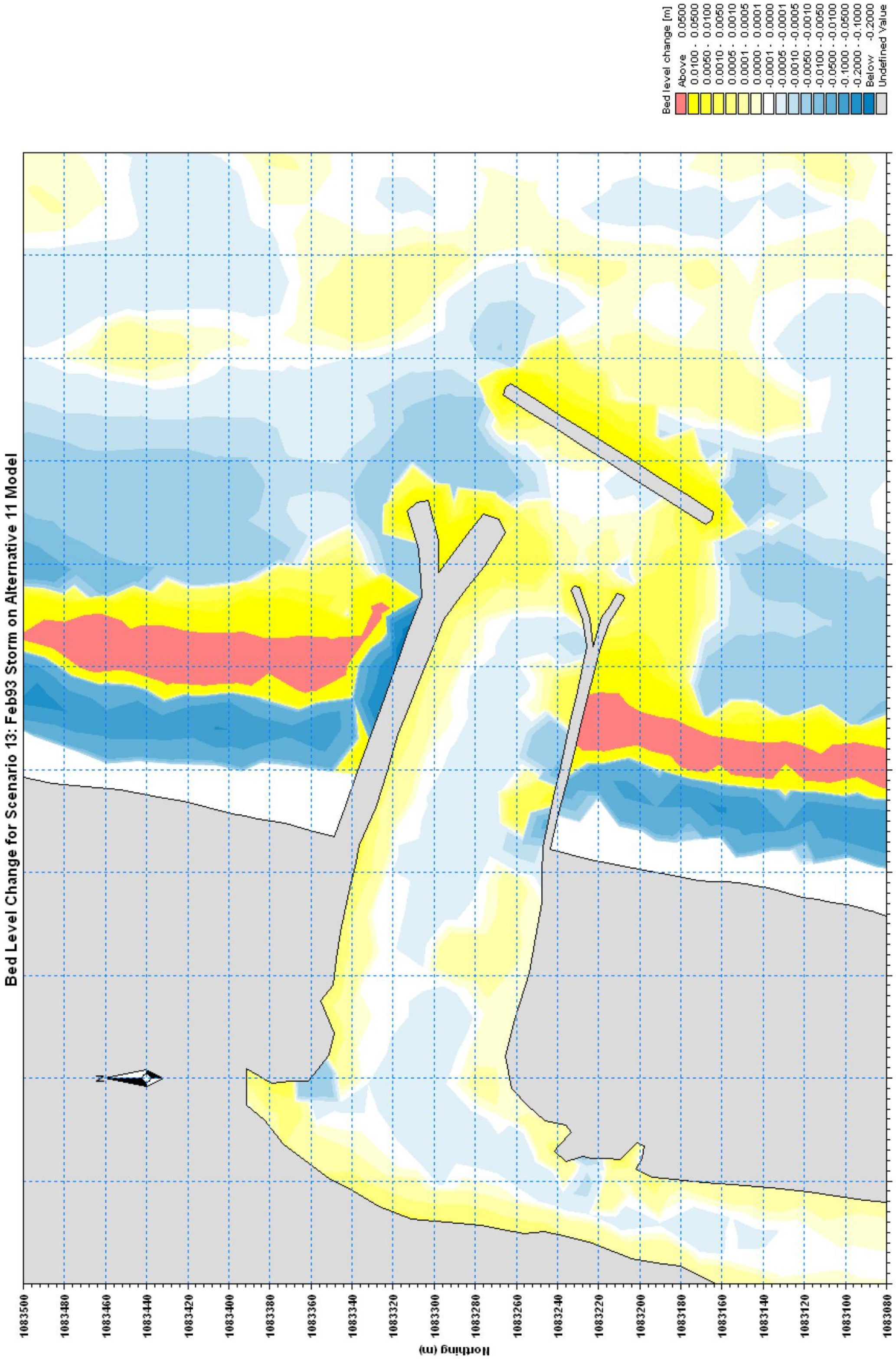


Exhibit 22h: Bed level changes for Alternative 11 Model Scenario 13.

The volume changes reported in Table 5 give an indication of the relative effectiveness of each alternative. Net erosion in some scenarios is an indication of the overwhelming predominance of scour over sedimentation. As explained earlier, net volume changes can be misleading because navigation-hindering sedimentation occurring in the channel fairway may not be reflected in this method of assessment. A comparison of net sedimentation for Scenarios 1 and 13 indicates that by implementing Alternative 11, sedimentation inside the inlet can be reduced by about 97%. Alternative 11 consisted of a detached 300 foot-long breakwater placed 165 feet (50 m) from the existing north jetty tip in order to facilitate navigation out of the inlet and around the structure. The specific location and actual dimensions of the structure would require further evaluation and optimization during preliminary engineering.

7. DISCUSSION OF THE PREFERRED ALTERNATIVE

While the erosion and sedimentation patterns shown in Exhibits 22a through 22h represent a qualitative overview of bed-level changes, a quantitative analysis was performed by extracting bed level changes across inlet cross-sections shown in Exhibit 19 for different scenarios. Exhibit 23 shows the cross-sectional changes for different scenarios of jetty configurations (5, 7, 8, 9, 10, 11, 12 and 13) together with the baseline case (Scenario 1). All of the sections have the same y-axis for easy comparison. The sections demonstrate that Scenario 13 represents the best alternative for reducing sedimentation within the inlet.

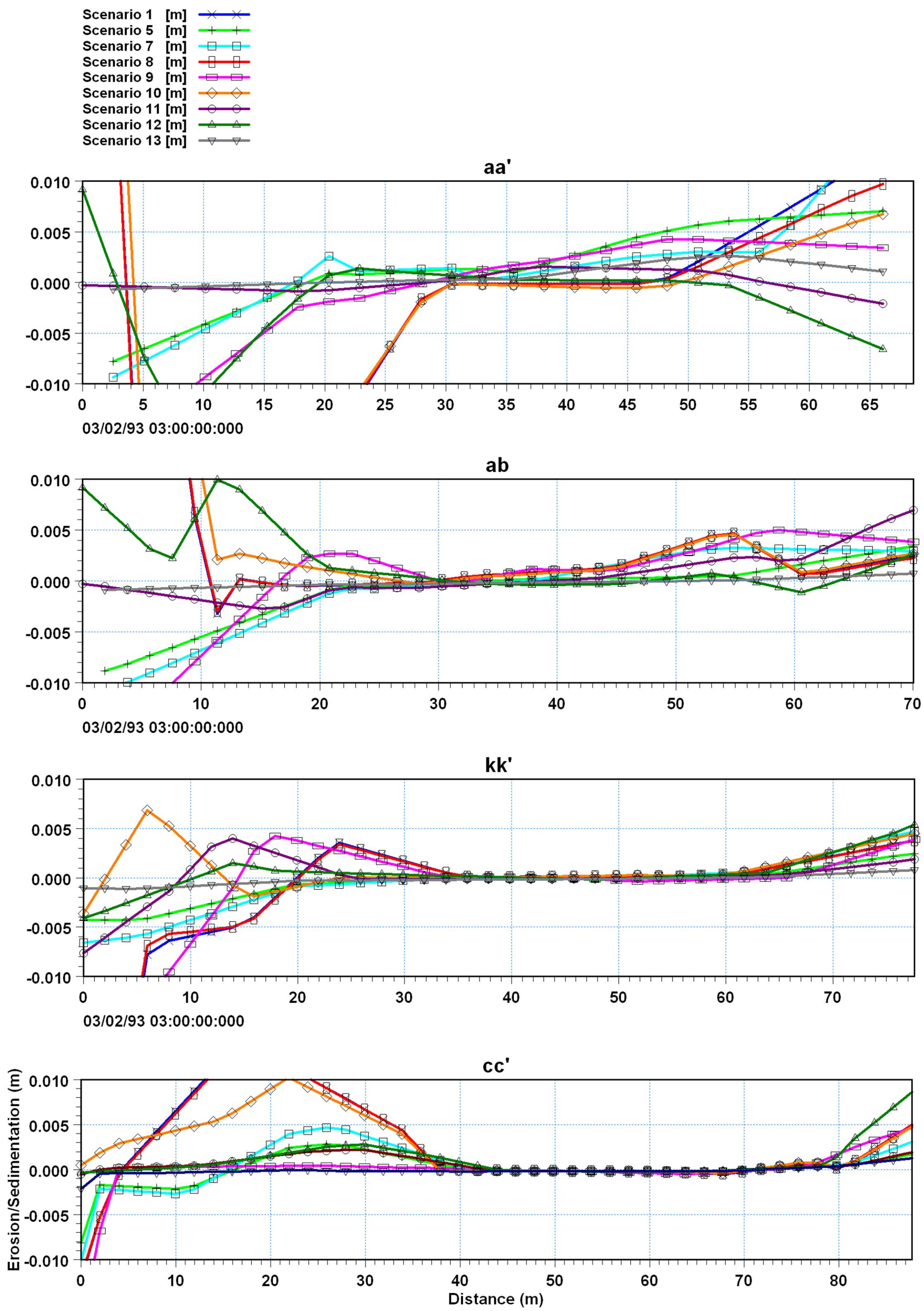


Exhibit 23:Bed level change cross-sections for scenarios 1, 5, 7, 8, 9, 10, 11, 12 and 13. a. Section a-a'. b. Section - ab. c. Section k-k'. d. Section c-c'.

Preliminary consultations with environmental agency representatives indicate that widening of the narrow-neck is not likely to be a feasible alternative because of permitting considerations associated with dredging significant areas of tidal wetlands. In Exhibit 24 the narrow-neck widening alternatives are eliminated, showing the sections for Scenarios 5, 7, 10 and 13 only together with the baseline case. Alternative 11 is the best option for minimizing sedimentation. The suitability of Alternative 11 can be seen further in section m-m' (Exhibit 25).

It is instructive to identify the causes of sedimentation and how Alternative 11 is effective in addressing those causes. In classical treatment of inlet sedimentation (O'Brien, 1931, 1971 and O'Brien and Dean, 1972), the equilibrium inlet cross-sectional area is found to be directly proportional to the tidal prism. For such a simple treatment of the problem, the proportionality coefficient varies from inlet to inlet representing different wave, hydraulic and sedimentary conditions. Stability of inlet entrances subjected to wave and tidal actions should be based on the relative magnitudes of tide and wave powers. Transport and deposition are the two opposing agents of sediment mobilization: while waves mobilize sediments, tidal currents transport and possibly flush out the accumulated sediments.

According to Bagnold's theory (1956) and longshore transport formulation (USACE, 1992), fluid power is the primary driving force for erosion and transport of sediments. For the Salt Ponds Inlet entrance, wave power is anticipated to be overwhelmingly higher than tidal power, which is supported by the low magnitude of tidal currents in the inlet. To demonstrate this, model results were used to show wave and tidal power at the inlet entrance for a tidal cycle during the February 1993 storm (Exhibit 26). Time-series data were extracted at point 'P' (Exhibit 19). This analysis indicated that wave power is about 2,000 times higher than tidal power (note, however, that only a fraction of this power is available near the bed for sediment transport). This explains why the model simulations indicate that the most significant sedimentation occurs near the inlet entrance.

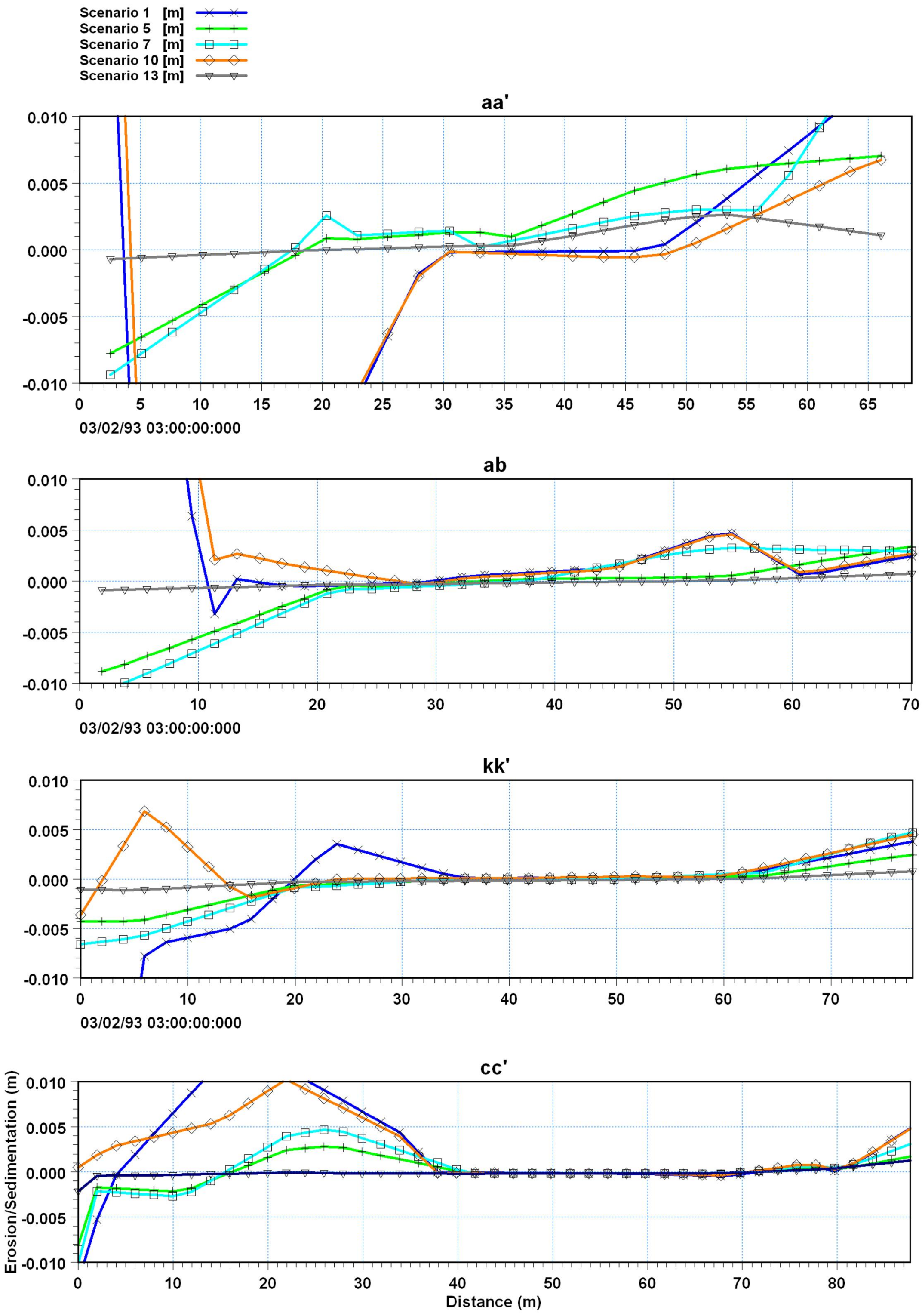


Exhibit 24: Bed level change cross-sections for scenarios 1, 5, 7, 10 and 13. a. Section a-a'. b. Section a-b. c. Section k-k'. d. Section c-c'.

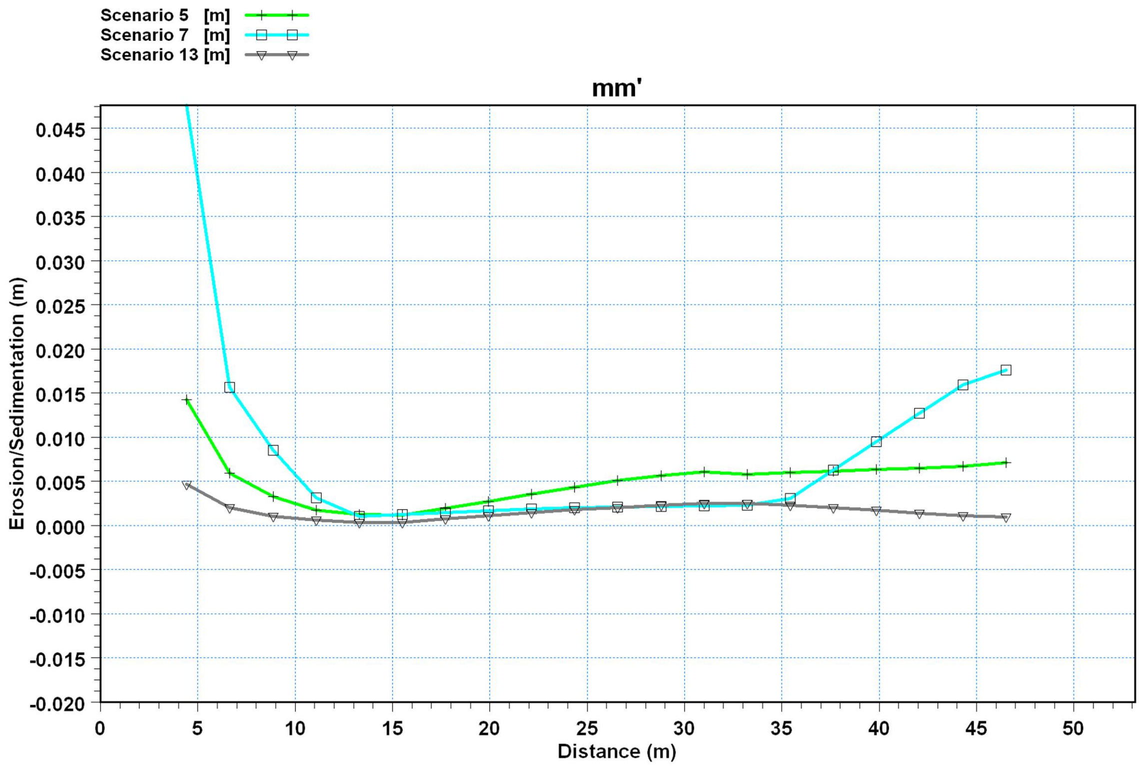


Exhibit 25: Bed level change cross-section m-m' for scenarios 5, 7 and 13.

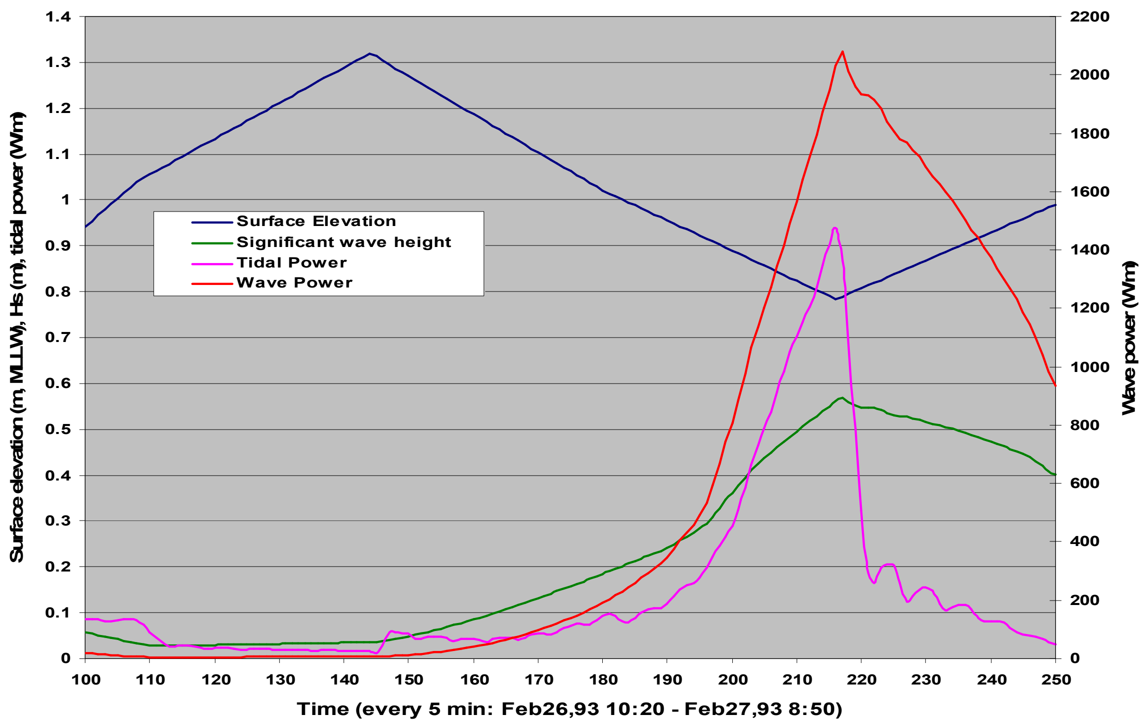


Exhibit 26: Wave and tidal power at the inlet entrance for As Is Scenario 1.

Given the predominance of wave power over tidal power at the Salt Ponds Inlet, there are two general conceptual approaches for protecting the inlet. One approach is to increase tidal power by increasing the tidal prism; however, this is not a feasible approach because it would require significantly enlarging the inlet. The other approach is to use engineered structures to decrease wave power entering the inlet. The alternatives included in this study were chosen with this goal in mind.

As noted above, the most effective single structure included in this study for the reduction of shoaling and dredging frequency is Alternative 11. In this alternative the offshore breakwater is elongated to 300 ft to provide complete protection from waves at the inlet entrance. Navigation is facilitated by openings at the north and south sides of the structure. Exhibit 27 shows the wave and tidal powers for the same location at ‘P’. Comparison of Exhibits 26 and 27 shows that Alternative 11 reduces wave power by about 96% from 2,000 W/m (waves per minute) to 70 W/m. The apparent reduction in tidal power shown in Exhibits 26 and 27 is possibly due to redistribution of currents across the channel section.

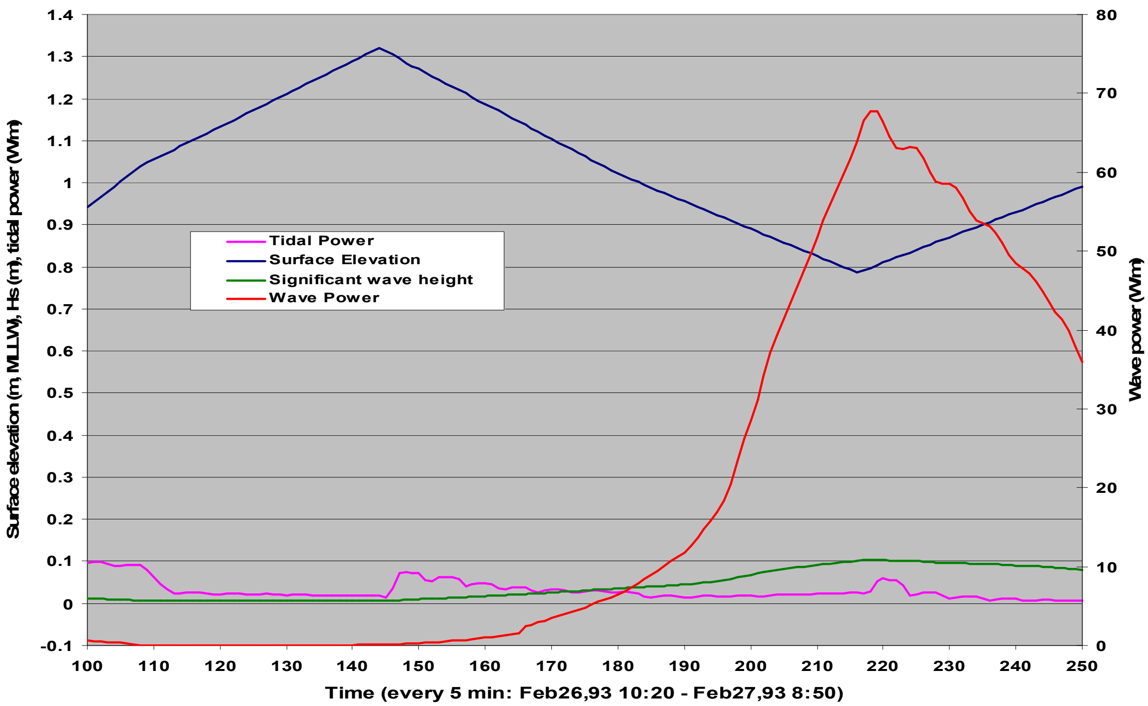


Exhibit 27: Wave and tidal power at the inlet entrance for Scenario 13.

Finally, a snapshot of simulated hydraulics is presented in Exhibit 28. Comparison of this Exhibit with Exhibit 16 demonstrates how the Alternative 11 changes flow and transport dynamics and reduces the ability of wind-waves to transport sediment into the inlet.

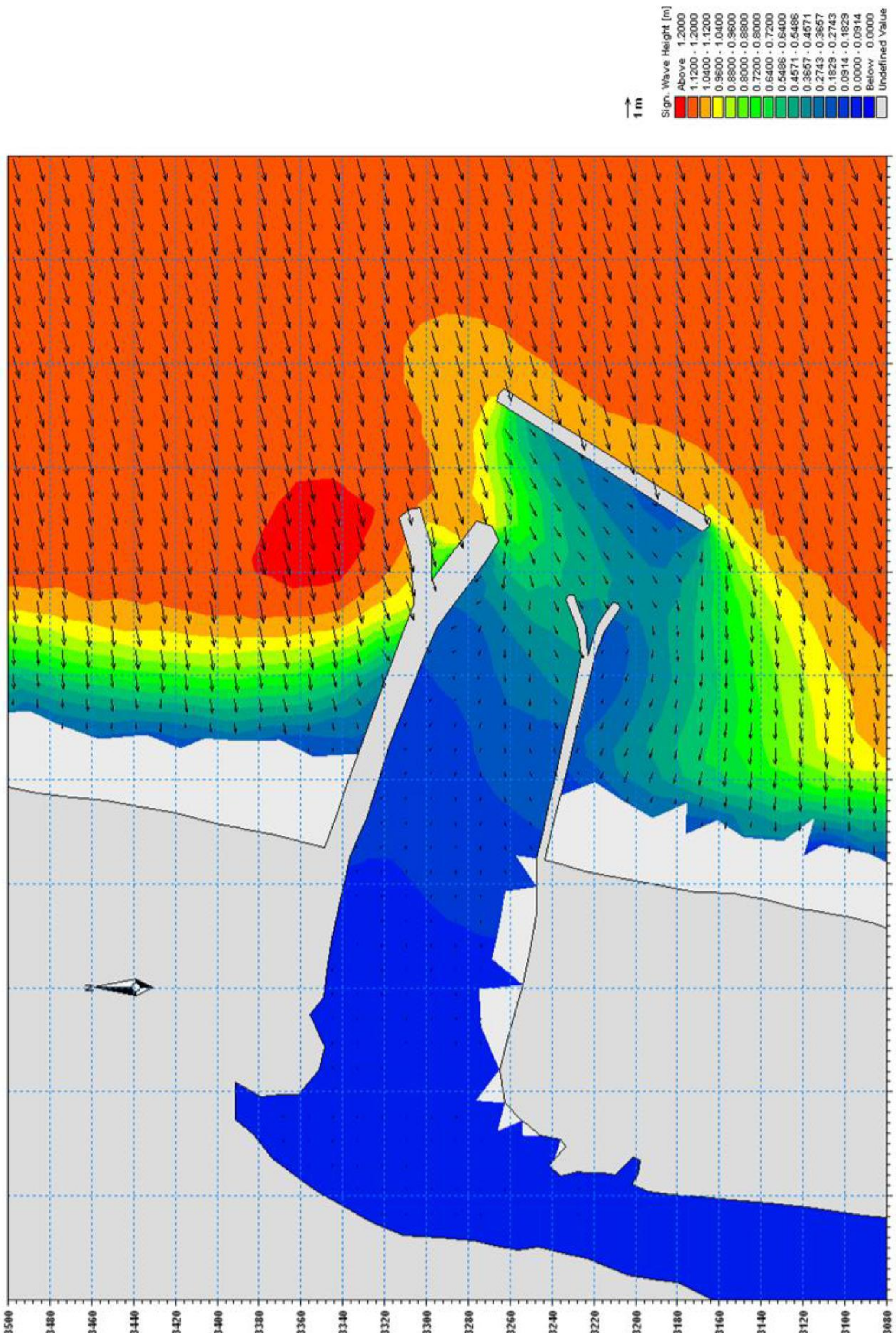


Exhibit 28a: Significant wave height snapshot for scenario 13 representing falling tide on February 27, 1993 at 12:15:00.

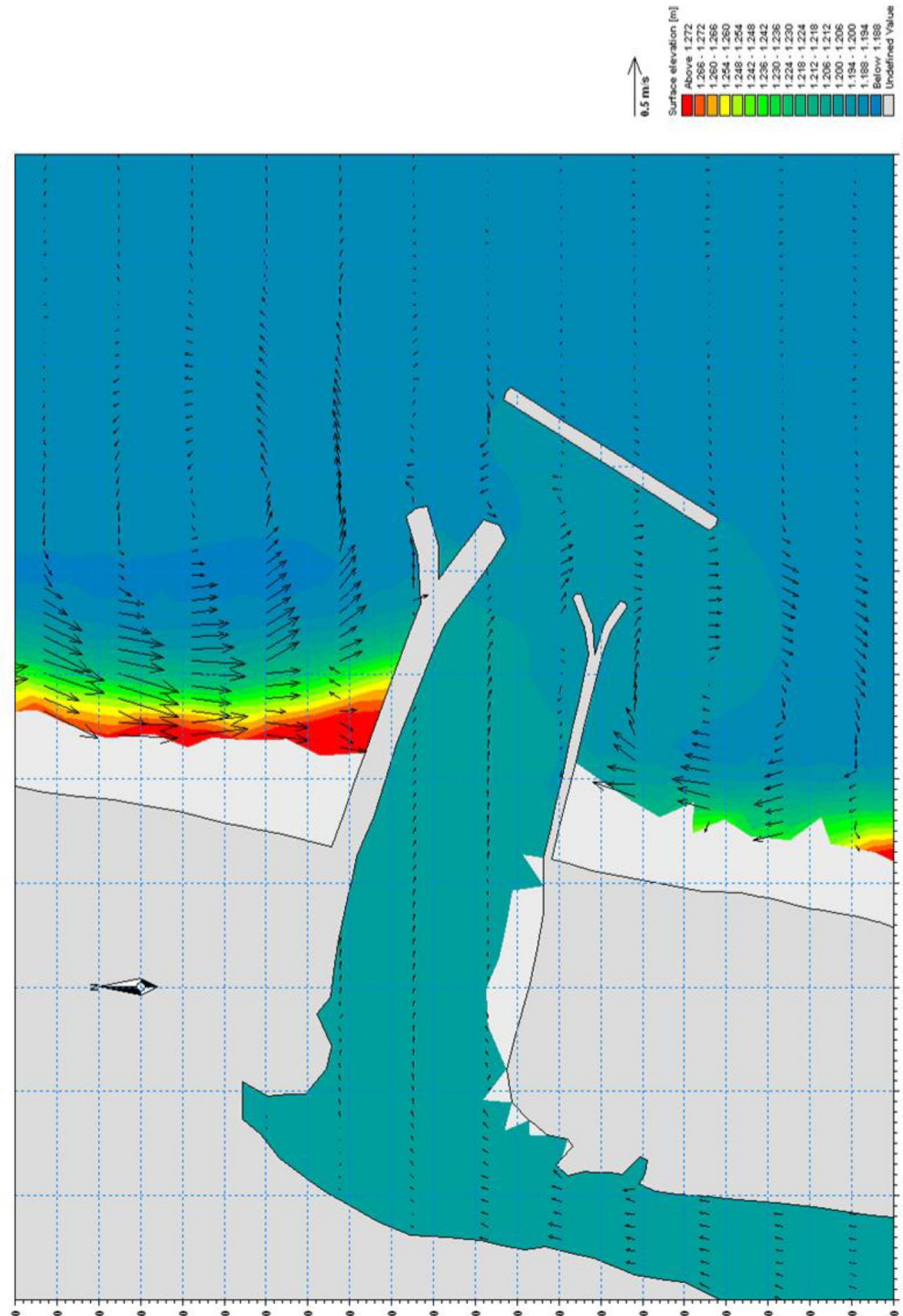


Exhibit 28b: Depth-averaged current snapshot for scenario 13 representing falling tide on February 27, 1993 at 12:15:00.

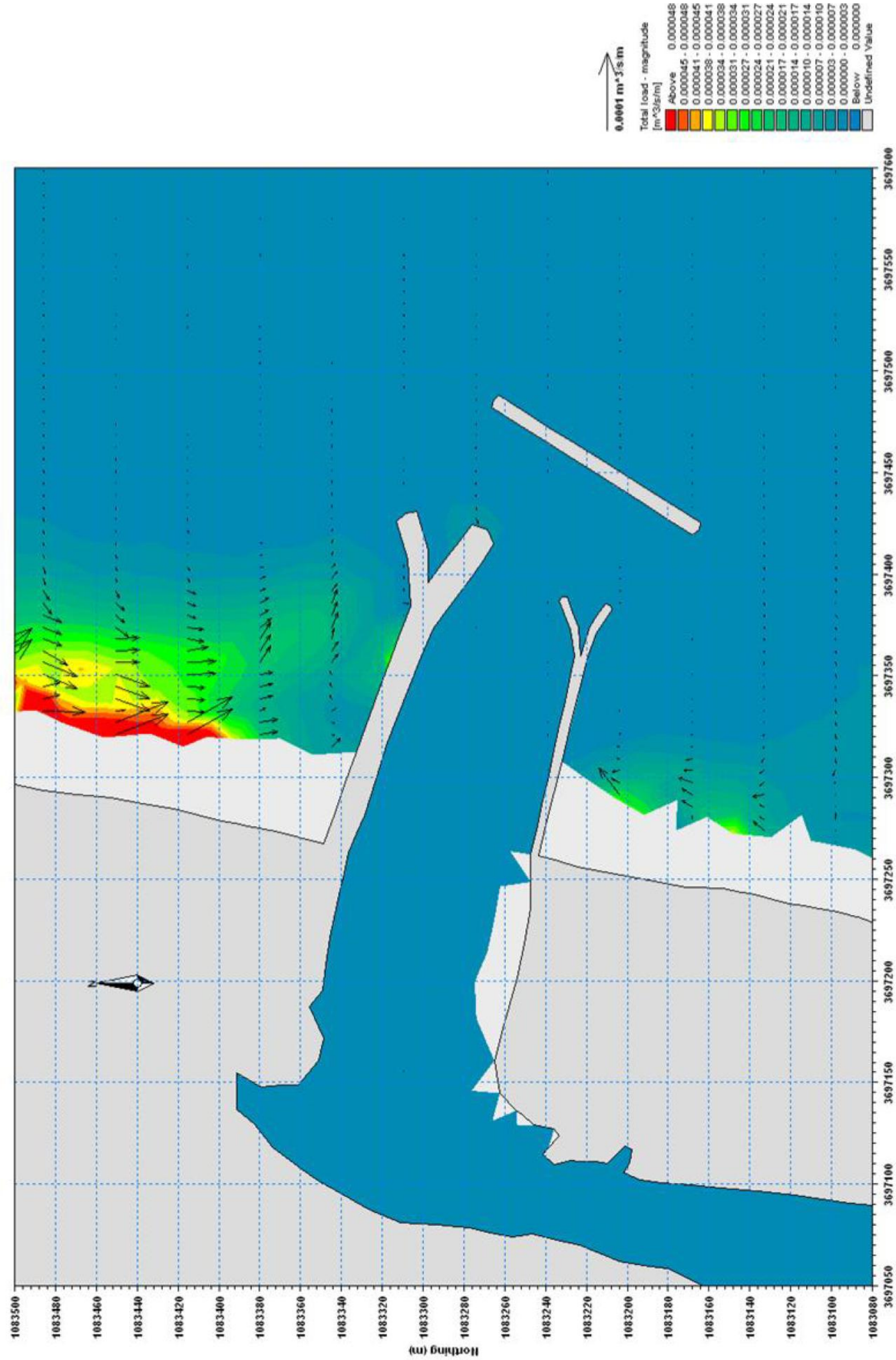


Exhibit 28c: Total load transport snapshot for scenario 13 representing falling tide on February 27, 1993 at 12:15:00.

8. SUMMARY AND RECOMMENDATIONS

8.1 Summary

Sediment transport measurements, computation and models suffer from large uncertainties (Soulsby, 1997). Model uncertainties are mostly a consequence of empirical transport formulae – the formulae proposed by different investigators hardly agree with each other. According to Davies et al (2002) computed sediment transports differ from the actual transports by a factor as great as 2 (2 times or 0.5 times?). With this note of caution about accepted uncertainties, the main findings of this study are summarized below.

Salt Ponds Inlet is a man-made inlet created in 1979 to connect Salt Ponds to the lower Chesapeake Bay. The north jetty was built at around the same time to help control sedimentation of the inlet; the original south jetty was an adapted timber groin that originally protected the adjacent south beach and was replaced by a vinyl sheetpile jetty in 2005. Since its creation, the inlet has experienced continual shoaling, perhaps in an attempt to return to its pre-inlet equilibrium condition. At present, the inlet requires dredging every 2 to 3 years.

As summarized in Table 6, the lower Chesapeake Bay near Salt Ponds is characterized by semi-diurnal tide, and wind-waves and swells.

Table 6: General environmental parameters

Characteristic parameter	**Description**
Tide	Semi-diurnal, mean tidal range = 2.43 ft (0.74 m)
Wave	Significant wave height: 50^{th} percentile = 0.66 ft (0.20 m) modal = 0.82 ft(0.25 m); 0.14% (12 hours/year) exceedence = 5.25 ft (1.6 m) Peak modal wave period: wind-wave = 5 s; swell = 9 s Major Direction: north-northeasterly; south-southwesterly
Bed-material sand	Representative median size: 0.205 mm; fine sand (Wentworth Scale)

A two-dimensional integrated model dynamically coupling wave, hydrodynamics and morphology was developed to simulate inlet processes and optimize proposed alternatives. The model domain encompassed the entire inlet, 2,000 ft (600 m) of shoreline, and an offshore boundary located at about 3,300 ft (1,000 m) seaward of the inlet entrance. A new 2009 bathymetric survey was used to define the model's computation grid. Jetties and breakwaters included in the simulations were modeled as solid, impermeable and non-overtoppable structures. Extra-tropical storms and Hurricane Isabel were used for forcing the model at the open boundaries.

No on-site measurements were dedicated to model calibration. Instead, relevant reported information was used to examine model performance. As summarized in Table 7, model results agreed well in orders of magnitude with the general available information on current, tidal prism, longshore transport and inlet shoaling rate at the inlet. In addition, wave diffraction around the breakwater was examined by sensitivity analysis.

Table 7: Model performance

Parameter	Reported	Computed
Current	0.26 ft/s (0.08 m/s), maximum	0 – 0.4 ft/s, (0 – 0.12 m/s) Feb93 storm 0 – 1.6 ft/s, (0 – 0.5 m/s) Sep03 Isabel
Tidal prism	~ 230,000 ft^3 (177,000 m^3)	Flux-based: ~ 160,500 ft^3 – 327,500 ft^3 (123,500 m^3 – 252,500 m^3)
Longshore transport	Shore loss rate north of the Inlet: 2,300 ft^3/year (1,766 m^3/year)	North of the Inlet, net : 18 to 52 ft^3 (14 to 40 m^3) for the 3.5-day February 1993 storm
Inlet shoaling rate	Bathymetry-based:~ 26,000 ft^3/year (20,000 m^3/year)	extrapolated: ~ 32,500 ft^3/year (25,000 m^3/year)

Model simulations indicated that the shoaling in the Salt Ponds Inlet is caused by both longshore and cross-shore transports. Under the present jetty configuration, cross-shore transport contribution to the shoaling problem appears to be significantly higher than longshore transport. This is perhaps one of the reasons why the Alternative 8 groin-field was not found to be effective for minimizing sedimentation in the inlet. There may also be contributions from rain-storm surface runoff from within the Salt Ponds catchment and aeolian transport of sediment, neither of which is accounted for in the present model.

Model results suggested that for the existing jetty configuration, inlet entrance wave power is about 2,000 times higher than the tidal power. This indicates the crux of the sedimentation problem: tidal action or prism does not have enough power to flush out sediments transported into the inlet by wave action.

A total of 13 scenarios were defined in order to examine 8 proposed alternatives, *As Is* and no jetties conditions (Table 4). The alternatives (Table 3) include various combinations of jetty modifications, narrow-neck widening, groin-field and jetty extension. Although narrow-neck widening requiring dredging in adjacent wetlands areas appeared very useful in minimizing sedimentation at the narrow-neck area, this option is anticipated to face difficulties related to environmental permitting concerns. Without the narrow-neck widening, sedimentation will continue to occur in that area (see Exhibits 22a, 22b, 22e, and 22h). However, sedimentation in this area is an order of magnitude lower than the inlet entrance area.

Examination of model-simulated aerial and cross-sectional bed-level changes suggests that Alternative 11 is the only measure that, by itself, is effective at minimizing shoaling in the inlet. Net bed level volume change (Table 5) suggests that implementation of this option would decrease sedimentation in the Inlet by 97%. Comparison of wave and tidal powers (Exhibits 26 and 27) suggests that peak wave powers would be reduced by 96% after implementation of Alternative 11. These two approaches indicate similar reduction rates. Accounting for sediment transport uncertainties, it is anticipated that Alternative 11 will reliably reduce inlet sedimentation by at least 50%. This means that the time between required maintenance dredging events can be expected to increase by a factor of at least 2 following implementation of Alternative 11, from the present 2-3 years to 4-6 years.

8.2 Findings, Recommendations and Opinions of Probable Cost

8.2.1 Findings

Alternative 11 is likely to significantly reduce sedimentation in the Salt Ponds Inlet, thus increasing the interval between maintenance dredging from the present 2-3 years to 4-6 years. If the City pursues this approach, optimization of Alternative 11 should occur during the preliminary engineering stage. Construction of a 300-foot long breakwater along with the installation of a rubble-mound structure over the existing south groin is expected to cost between roughly \$3 and \$3.5 million in 2010 dollars. Consultations with boaters indicate that the breakwater could be positioned and constructed such that adequate navigational access to the inlet entrance could still be maintained around the structure.

8.2.2 Recommendations

Based on the simulation results of the modifications included in this study and discussions with the waterway users and the City, KHA recommends the following measures be undertaken:

1. Develop plans for raising, armoring and extending the existing south timber/vinyl sheetpile jetty by construction of a rubble-mound structure over the existing jetty. The anticipated cost of constructing the rubble-mound structure over the existing south jetty to a uniform elevation of 9 feet above MLW is approximately \$1 million. Although the hydrodynamic model did not indicate a significant change in shoaling patterns would be gained by this structure, it is expected to have a marginally beneficial effect on reducing shoaling in the inlet by preventing periodic overtopping of the structure.
2. Further evaluate the optimal size, location, configuration, and cost of an offshore breakwater at the mouth of the inlet. Additional investigations, including more advanced modeling efforts and preliminary design should be undertaken to gain more information on the most effective location and configuration for the structure as well as the shoaling pattern around such a structure, and its orientation so as not to unreasonably interfere with navigation into and out of the inlet. The northern and southern model boundaries should be extended. This would facilitate an analysis of the efficacy of suitably spaced (both cross-shore and longshore dimensions) offshore breakwaters in stabilizing the shoreline as well as minimizing sedimentation of the inlet. The larger modeling domain will lend to greater degree of confidence in the analysis at local points of investigation. However, any decision regarding the location and size of the structure should be coordinated with the results of a proposed study regarding the stability of the Hampton Shoreline in the Buckroe and Grandview areas in order to more effectively assess system-wide influences and address the concerns of the various Hampton communities affected by coastal processes in a comprehensive manner.
3. The option of narrow-neck widening should not be ruled out if environmental permitting concerns can be effectively addressed. The recommended Alternative 11 (offshore breakwater with south groin extension and hardening) will not reduce the shoaling rate in the narrow-neck area since shoaling hydraulics are different there than at the inlet entrance. Although the shoaling rate at the narrow-neck is lower than at the inlet entrance by an order of magnitude it is still of concern to the users of the Salt Ponds Inlet and could be lessened significantly by widening of the inlet in this area.

In addition to the structural measures above, the following additional non-structural management measures are recommended to evaluate the effectiveness of any structure undertaken and better document maintenance dredging costs:

1. A dye study should be undertaken to determine if sand migration through the north jetty is a significant source of shoaling within the inlet. If so, the preliminary design of the san tightening measures previously prepared should be advanced to final design and a cost estimate for the tightening prepared. If undertaken, an examination of the effectiveness of extending the north jetty and re-orienting the mouth of the Inlet to an east-southeast direction should be undertaken as a means of further protecting the inlet from wave action from the east.

2. The City should install upward looking Acoustic Doppler Current Profilers (ADCPs) twice annually for a 30-day period at the offshore model boundary and near the inlet entrance in order to simultaneously measure water level, current and wave properties. The measurements should be accompanied by pre- and post-deployment bathymetric surveys. These measurements would help analysis during the preliminary engineering stage.
3. The City should continue, and consider expanding the annual hydrographic surveys of the Salt Ponds Inlet.
4. The City should survey and monitor shoreline and inlet cross-sectional profiles at suitable intervals in order to capture seasonal morphological trends. This will facilitate future analysis and modeling efforts and lead to improvements in the structure(s) selected for design and construction. A comprehensive survey and monitoring effort will help explain both seasonal and episodic events and their relative impacts on the performance of the inlet and adjacent shorelines
5. The City should institute a formalized system of record-keeping and survey work to document dredge volumes and beach morphology along the south beach where sediments have been routinely placed during maintenance dredging events.
6. The City should examine whether an On-Call contract for dredging the Salt Ponds and other city waterways or creation of a City dredging capability could help contain or reduce long term dredging costs.

8.2.3 Opinions of Probable Cost

The anticipated costs of the structural alternatives considered above are as follows:

- Raising and extending the south groin – approximately $1 million
- Construction of a 300 foot long offshore breakwater – approximately $2.0 to 2.5 million
- Construction of the improvements to the south groin and installation of the offshore breakwater together $3.0 – 3.5 million.

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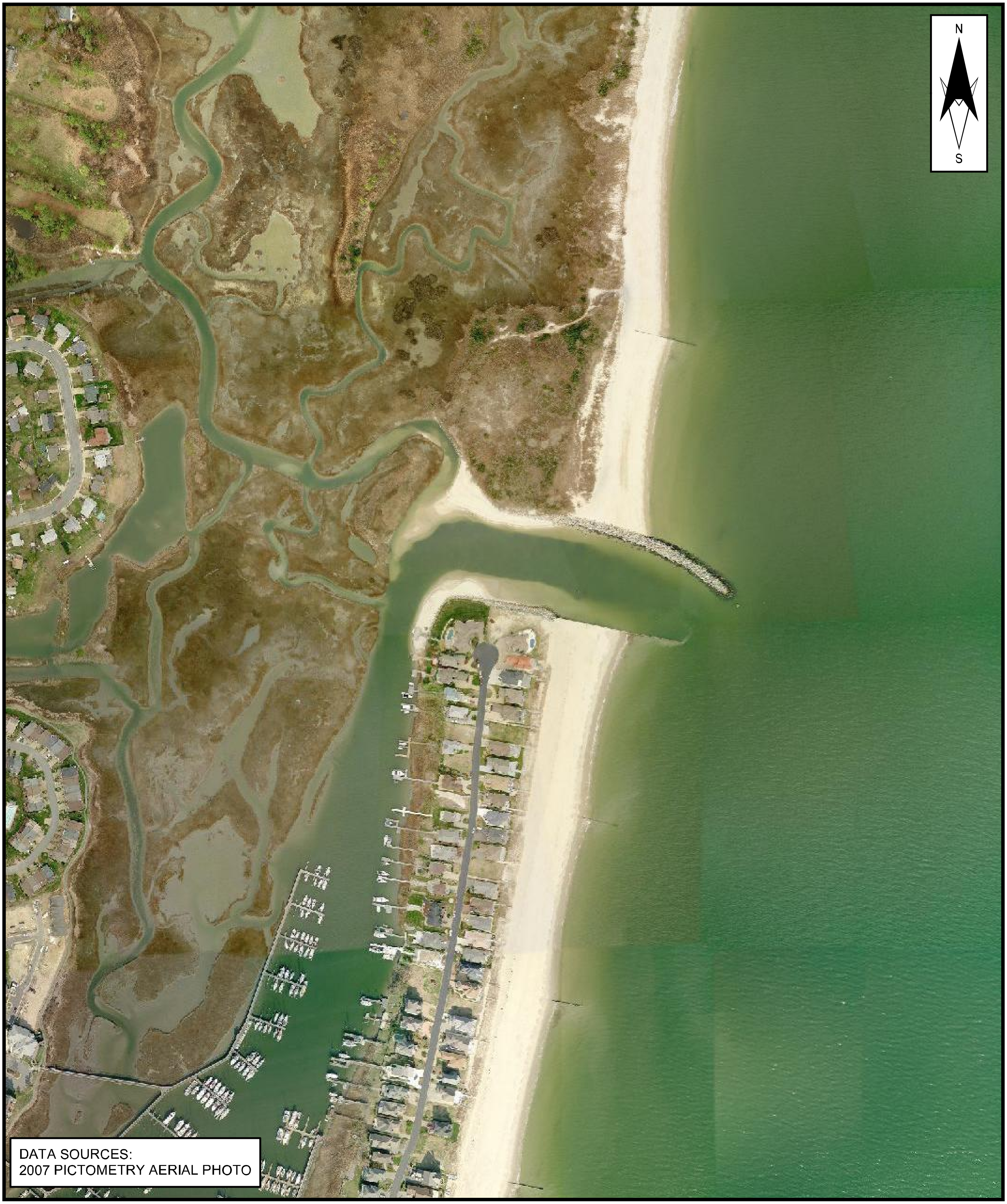
APPENDIX A

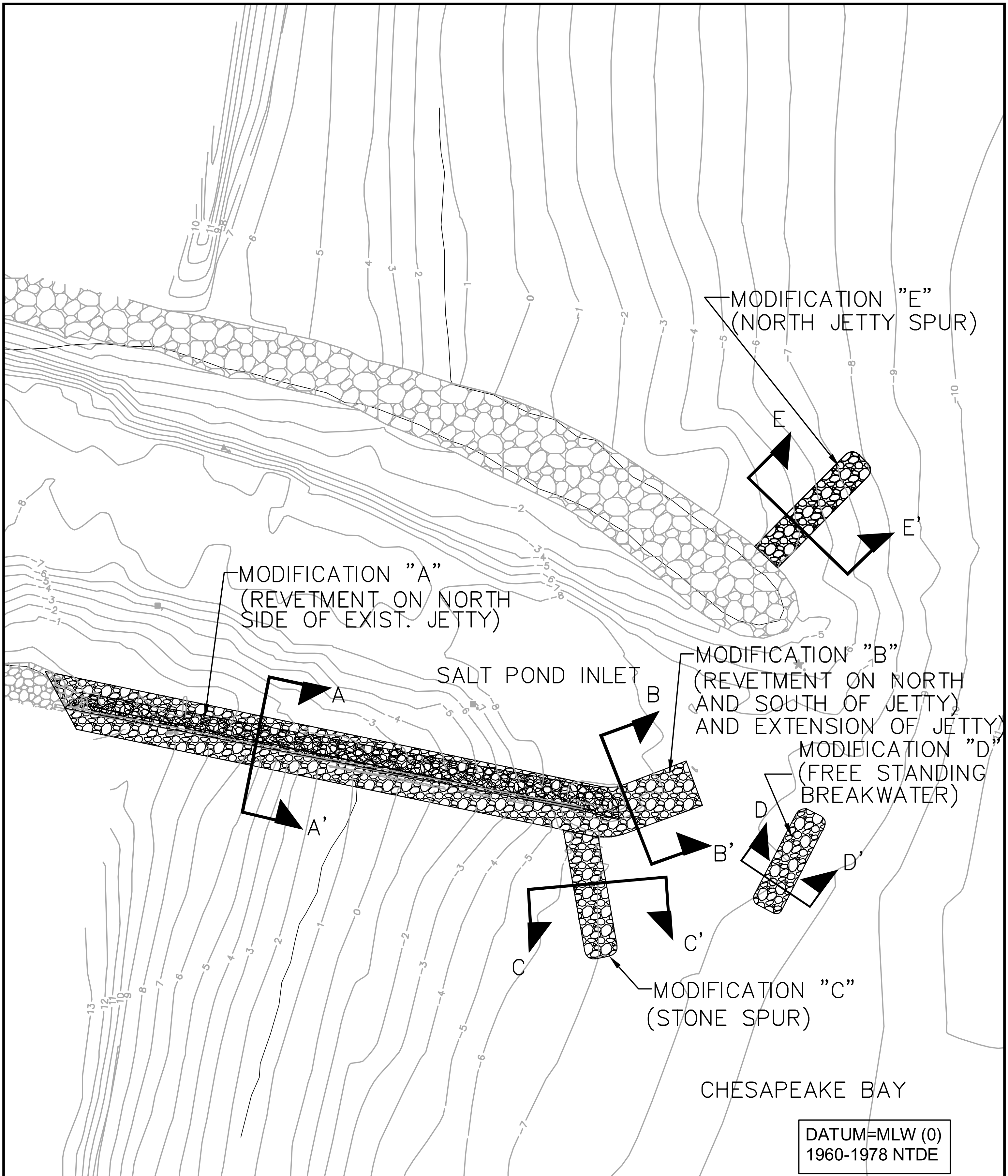
SUPPLEMENTARY FIGURES

Kimley-Horn and Associates, Inc.	INLET MANAGEMENT PLAN SALT PONDS HAMPTON, VIRGINIA	**FIGURE 1 SALT PONDS LOCATION MAP**
	CREATED BY: PC — SCALE:1 IN = 4,000 FT	
	DATE: 2-1-2010 — KHA PROJECT NUMBER: 116227018	



Kimley-Horn and Associates, Inc.

INLET MANAGEMENT PLAN SALT PONDS HAMPTON, VIRGINIA		FIGURE 2 SALT PONDS INLET AERIAL PHOTO MAP
CREATED BY: PC	SCALE:1 IN = 400 FT	
DATE: 2-1-2010	KHA PROJECT NUMBER: 116227018	



SCALE (H):	1" = 100'
SCALE (V):	NONE
DESIGNED BY:	PC
DRAWN BY:	JM
DATE:	11/24/09
KHA PROJECT NO.: 116227018	

FIGURE 3

SALT PONDS INLET MANAGEMENT PLAN

SIMULATED MODIFICATIONS

CITY OF HAMPTON DEPARTMENT OF PUBLIC WORKS

HAMPTON **VIRGINIA**

Kimley-Horn and Associates, Inc.
Suite 300
501 Independence Parkway
Chesapeake, Virginia 23320
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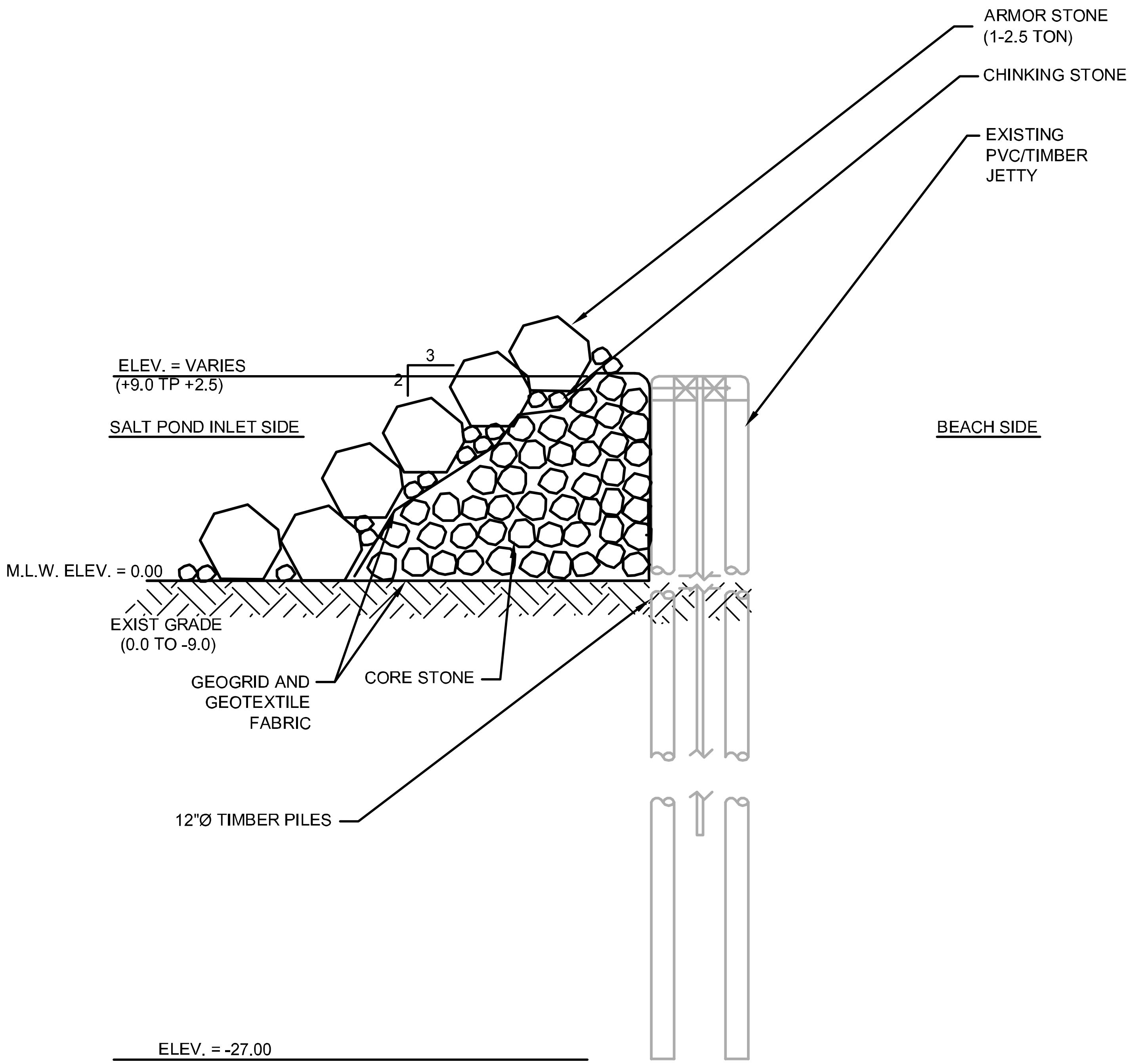
Fax: (757) 548-7301



SECTION A-A' SOUTH JETTY MODIFICATION

NOT TO SCALE

DATUM=MLW (0)
1960-1978 NTDE

SCALE (H):	AS SHOWN
SCALE (V):	NONE
DESIGNED BY:	PC
DRAWN BY:	JM
DATE:	11/24/09
KHA PROJECT NO.: 116227018	
FIGURE 4	

SALT PONDS INLET MANAGEMENT PLAN

MODIFICATION A SOUTH JETTY

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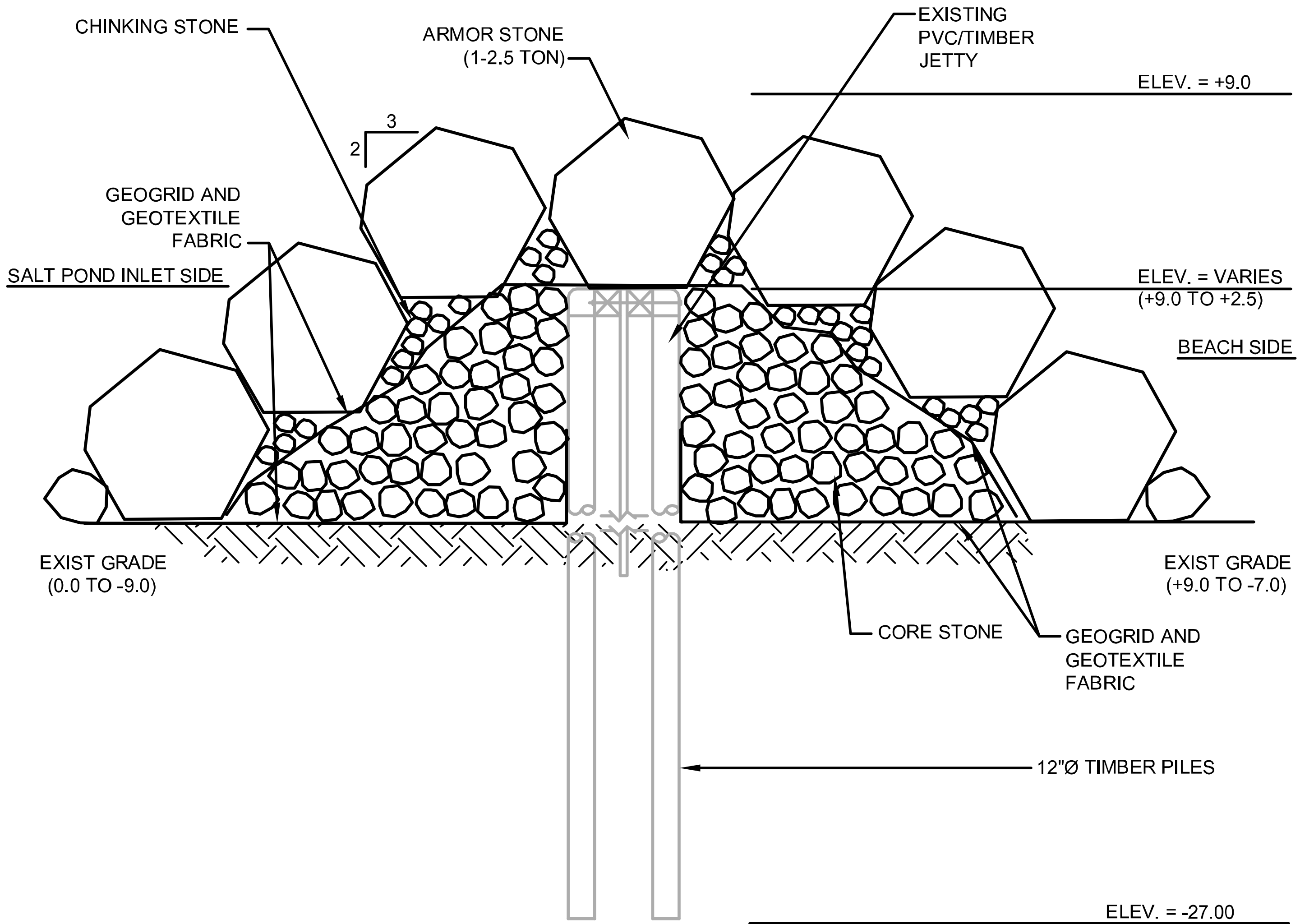

SECTION B-B' SOUTH JETTY MODIFICATION

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1960-1978 NTDE

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SCALE (V):	NONE
DESIGNED BY:	PC
DRAWN BY:	JM
DATE:	11/24/09
KHA PROJECT NO.: 116227018	
FIGURE 5	

SALT PONDS INLET MAINTENANCE PLAN

MODIFICATION B SOUTH JETTY

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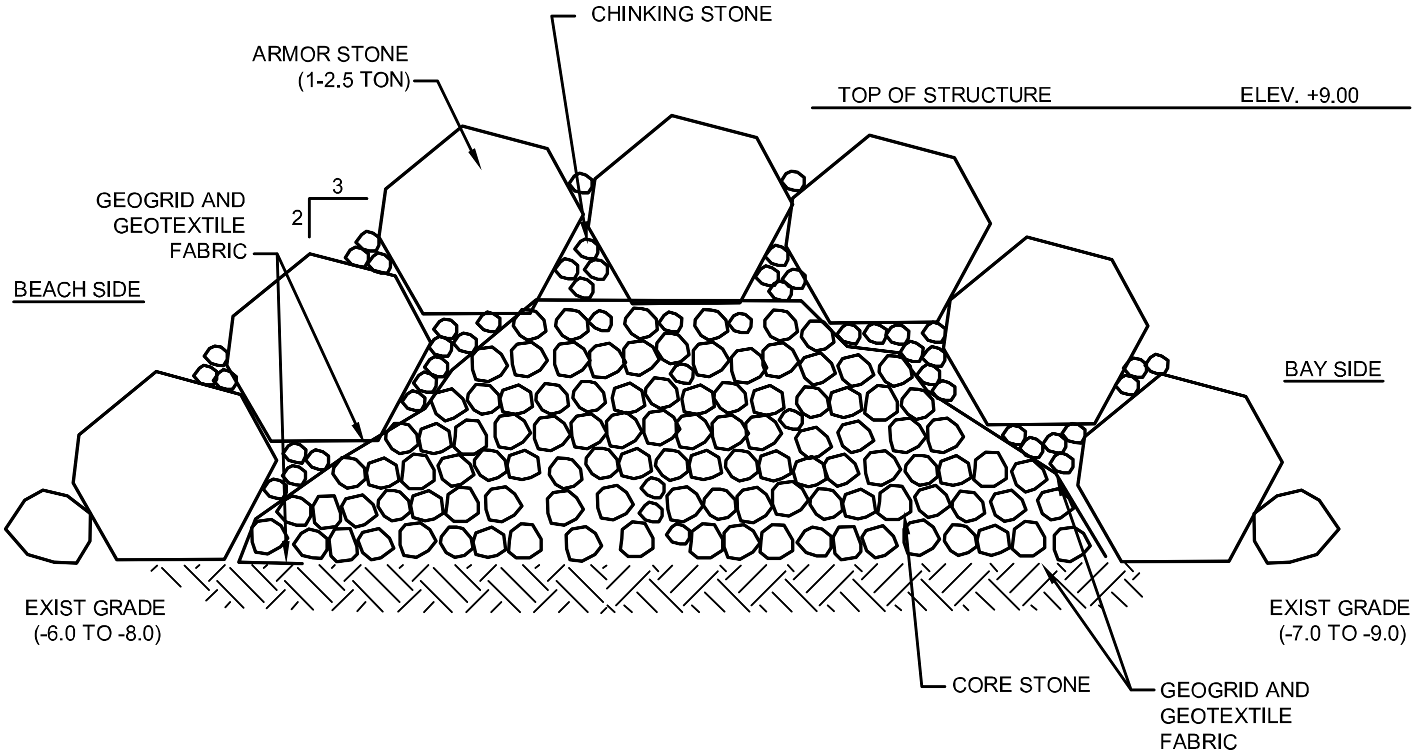

SECTION C-C' SOUTH JETTY SPUR

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1960-1978 NTDE

SCALE (H):	AS SHOWN
SCALE (V):	NONE
DESIGNED BY:	PC
DRAWN BY:	JM
DATE:	1/24/09
KHA PROJECT NO.: 116227018	
FIGURE 6	

SALT PONDS INLET MAINTENANCE PLAN

MODIFICATION C SOUTH JETTY SPUR

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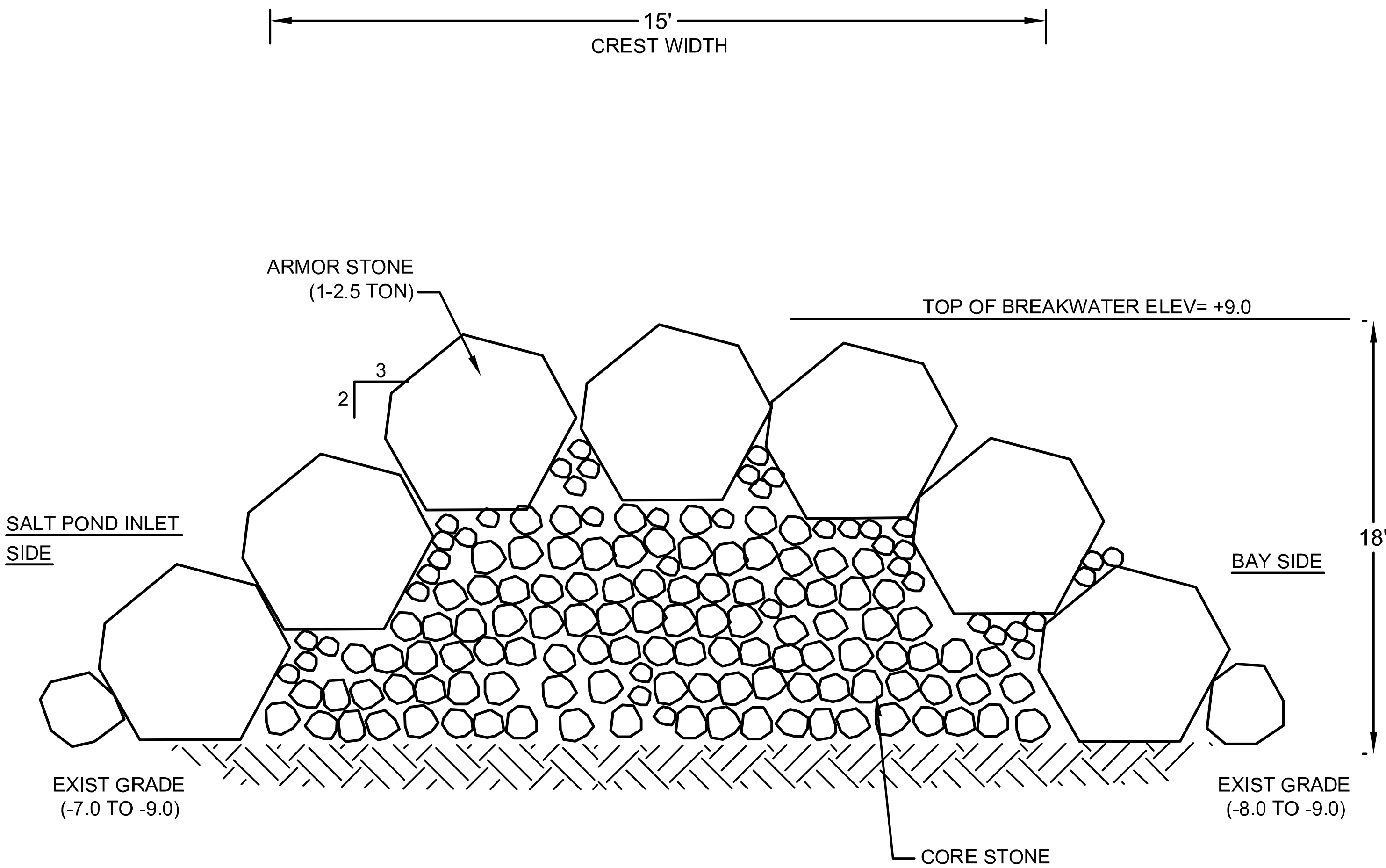

SECTION D-D' DETACHED BREAKWATER

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1960-1978 NTDE

SCALE (H):	AS SHOWN
SCALE (V):	NONE
DESIGNED BY:	PC
DRAWN BY:	JM
DATE:	1/24/09
KHA PROJECT NO.: 116227018	
FIGURE 7	

SALT PONDS INLET MAINTENANCE PLAN

MODIFICATION D DETACHED BREAKWATER

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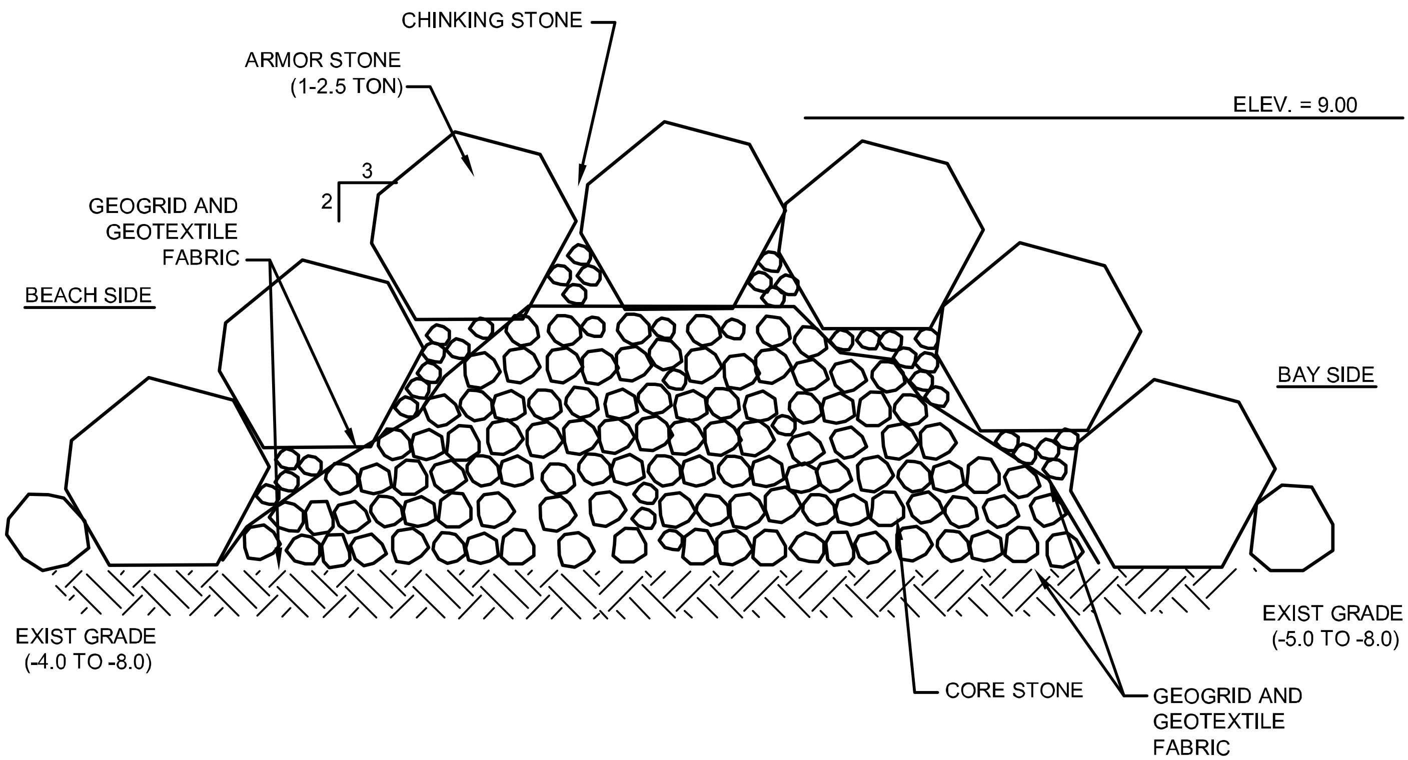

SECTION E-E NORTH JETTY SPUR

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1960-1978 NTDE

SCALE (H):	AS SHOWN
SCALE (V):	NONE
DESIGNED BY:	PC
DRAWN BY:	JM
DATE:	11/24/09
KHA PROJECT NO.: 116227018	
FIGURE 8	

SALT PONDS INLET MAINTENANCE PLAN

MODIFICATION E NORTH JETTY SPUR

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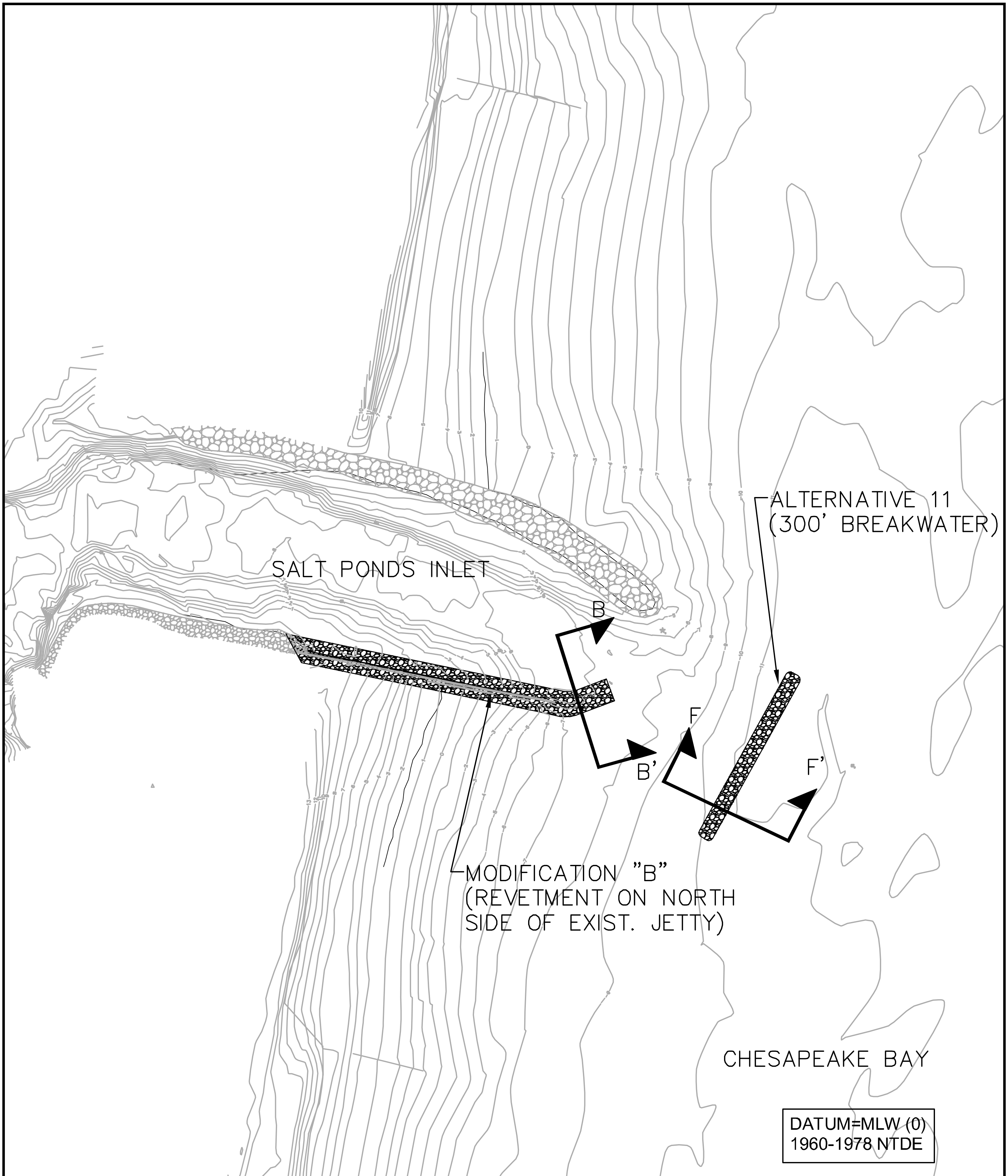

SCALE (H):	1" = 200'
SCALE (V):	NONE
DESIGNED BY:	PC
DRAWN BY:	JM
DATE:	11/24/09
KHA PROJECT NO.: 116227018	

FIGURE 9

SALT PONDS INLET MANAGEMENT PLAN

ALTERNATIVE 11

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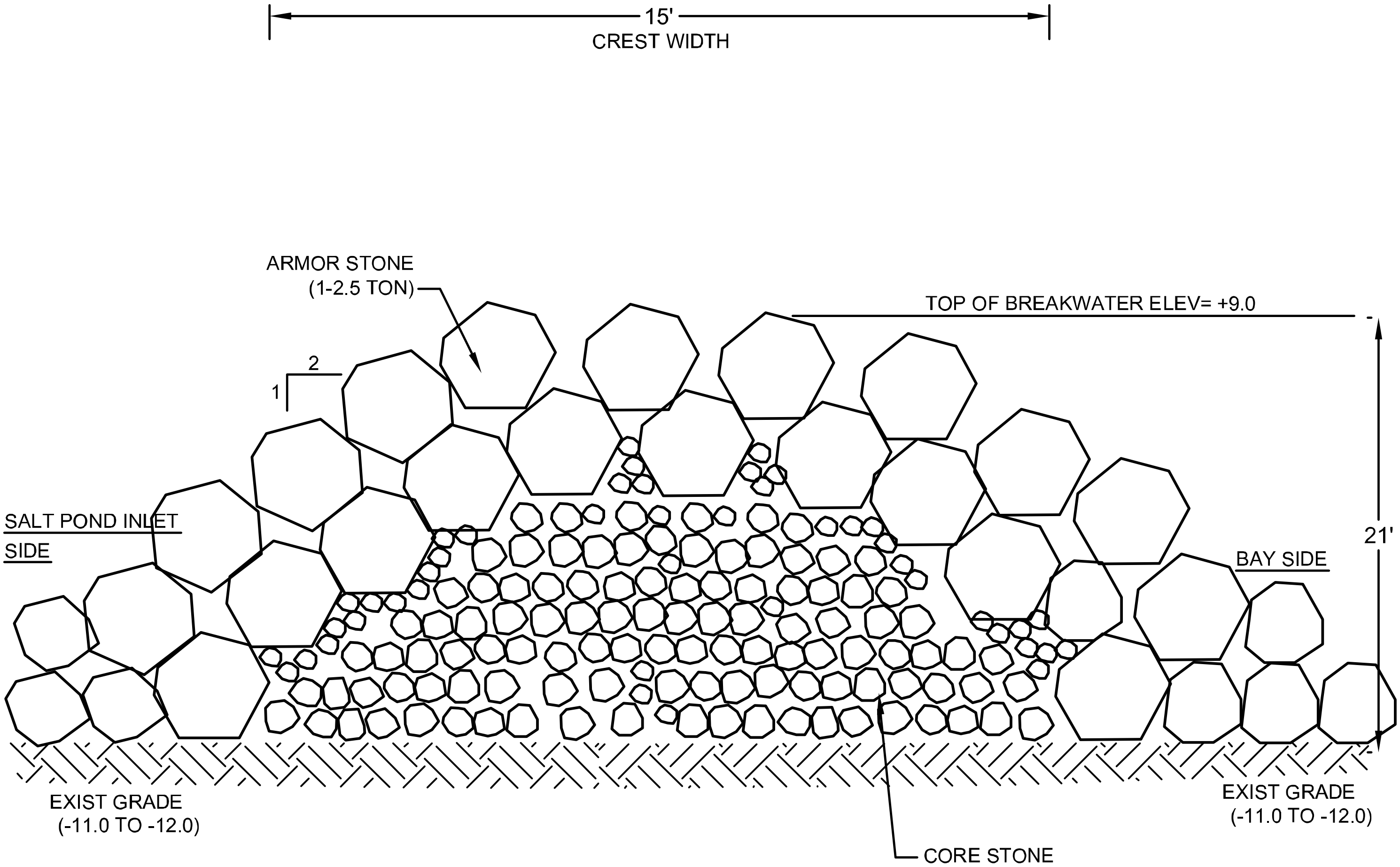

SECTION F-F' DETACHED 300' BREAKWATER

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SCALE (V):	NONE
DESIGNED BY:	PC
DRAWN BY:	JM
DATE:	1/24/09
KHA PROJECT NO.: 116227018	
FIGURE 10	

SALT PONDS INLET MAINTENANCE PLAN

MODIFICATION F DETACHED BREAKWATER

CITY OF HAMPTON DEPARTMENT OF PUBLIC WORKS

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